

**U.S. DEPARTMENT OF COMMERCE
National Technical Information Service**

AD-A028 031

State-of-the-Art Review on Shotcrete

California Univ.

**Prepared For
Army Engineer Waterways Experiment Station**

June 1976

KEEP UP TO DATE

Between the time you ordered this report—which is only one of the hundreds of thousands in the NTIS information collection available to you—and the time you are reading this message, several *new* reports relevant to your interests probably have entered the collection.

Subscribe to the **Weekly Government Abstracts** series that will bring you summaries of new reports as soon as they are received by NTIS from the originators of the research. The WGA's are an NTIS weekly newsletter service covering the most recent research findings in 25 areas of industrial, technological, and sociological interest—invaluable information for executives and professionals who must keep up to date.

The executive and professional information service provided by NTIS in the **Weekly Government Abstracts** newsletters will give you thorough and comprehensive coverage of government-conducted or sponsored re-

search activities. And you'll get this important information within two weeks of the time it's released by originating agencies.

WGA newsletters are computer produced and electronically photocomposed to slash the time gap between the release of a report and its availability. You can learn about technical innovations immediately—and use them in the most meaningful and productive ways possible for your organization. Please request NTIS-PR-205/PCW for more information.

The weekly newsletter series will keep you current. But *learn what you have missed in the past* by ordering a computer **NTISearch** of all the research reports in your area of interest, dating as far back as 1964, if you wish. Please request NTIS-PR-186/PCN for more information.

WRITE: Managing Editor
5285 Port Royal Road
Springfield, VA 22161

Keep Up To Date With SRIM

SRIM (Selected Research in Microfiche) provides you with regular, automatic distribution of the complete texts of NTIS research reports *only* in the subject areas you select. SRIM covers almost all Government research reports by subject area and/or the originating Federal or local government agency. You may subscribe by any category or subcategory of our WGA (**Weekly Government Abstracts**) or **Government Reports Announcements and Index** categories, or to the reports issued by a particular agency such as the Department of Defense, Federal Energy Administration, or Environmental Protection Agency. Other options that will give you greater selectivity are available on request.

The cost of SRIM service is only 45¢ domestic (60¢ foreign) for each complete

microfiched report. Your SRIM service begins as soon as your order is received and processed and you will receive biweekly shipments thereafter. If you wish, your service will be backdated to furnish you microfiche of reports issued earlier.

Because of contractual arrangements with several Special Technology Groups, not all NTIS reports are distributed in the SRIM program. You will receive a notice in your microfiche shipments identifying the exceptionally priced reports not available through SRIM.

A deposit account with NTIS is required before this service can be initiated. If you have specific questions concerning this service, please call (703) 451-1558, or write NTIS, attention SRIM Product Manager.

This information product distributed by

NTIS

U.S. DEPARTMENT OF COMMERCE
National Technical Information Service
5285 Port Royal Road
Springfield, Virginia 22161

Unclassified

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

REPORT DOCUMENTATION PAGE		READ INSTRUCTIONS BEFORE COMPLETING FORM
1. REPORT NUMBER Contract Report S-76-4	2. GOVT ACCESSION NO.	3. RECIPIENT'S CATALOG NUMBER AD-AC28031
4. TITLE (and Subtitle) STATE-OF-THE-ART REVIEW ON SHOTCRETE		5. TYPE OF REPORT & PERIOD COVERED Final report
		6. PERFORMING ORG. REPORT NUMBER
7. AUTHOR(s) T. L. Brekke, H. H. Einstein, USBR Engineering and Research Center, and R. E. Mason		8. CONTRACT OR GRANT NUMBER(s) DACA 39-74-M-0371 ✓ DACA 39-75-M-0082 ✓
9. PERFORMING ORGANIZATION NAME AND ADDRESS U. of Calif., Berkeley, Calif. MIT, Cambridge, Mass. USBR Engineering and Research Center, Denver, Colo. A. A. Mathews, Inc., Rockville, Md.		10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS
11. CONTROLLING OFFICE NAME AND ADDRESS Office, Chief of Engineers, U. S. Army Washington, D. C. 20314		12. REPORT DATE June 1976
14. MONITORING AGENCY NAME & ADDRESS (if different from Controlling Office) U. S. Army Engineer Waterways Experiment Station Soils and Pavements Laboratory P. O. Box 631, Vicksburg, Miss. 39180		13. NUMBER OF PAGES 122
		15. SECURITY CLASS. (of this report) Unclassified
		15a. DECLASSIFICATION/DOWNGRADING SCHEDULE
16. DISTRIBUTION STATEMENT (of this Report) Approved for public release; distribution unlimited.		
17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report)		
18. SUPPLEMENTARY NOTES REPRODUCED BY NATIONAL TECHNICAL INFORMATION SERVICE U. S. DEPARTMENT OF COMMERCE SPRINGFIELD, VA. 22161		
19. KEY WORDS (Continue on reverse side if necessary and identify by block number) New Austrian Tunneling Method Shotcrete State-of-the-art studies Tunnel linings		
20. ABSTRACT (Continue on reverse side if necessary and identify by block number) This report contains the results of four studies pertaining to shotcrete technology. The first part of the report is an overview of American practices in shotcrete. The second part is a discussion of the development and practice of shotcrete technology in Europe, including a description of the New Austrian Tunneling Method (NATM). The third part describes the experiences of the U. S. Bureau of Reclamation in using shotcrete for tunnel linings and their design and construction concepts. The final part of the report is a discussion of instrumentation currently employed for obtaining the data used in (Continued)		

DD FORM 1473
1 JAN 73

EDITION OF 1 NOV 65 IS OBSOLETE

REPRODUCED BY
NATIONAL TECHNICAL
INFORMATION SERVICE
U. S. DEPARTMENT OF COMMERCE
SPRINGFIELD, VA. 22161

Unclassified

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

SECURITY CLASSIFICATION OF THIS PAGE(When Data Entered)

design of shotcreted tunnels being constructed by the NATM. The NATM involves the use of the theoretical-observational approach to design. Each part of the report also gives recommendations as to needed research requirements and suggestions pertaining to possible extended uses of shotcrete in this country.

A

SECURITY CLASSIFICATION OF THIS PAGE(When Data Entered)

THE CONTENTS OF THIS REPORT ARE NOT TO BE
USED FOR ADVERTISING, PUBLICATION, OR
PROMOTIONAL PURPOSES. CITATION OF TRADE
NAMES DOES NOT CONSTITUTE AN OFFICIAL EN-
DORSEMENT OR APPROVAL OF THE USE OF SUCH
COMMERCIAL PRODUCTS.

PREFACE

This study was conducted by the U. S. Army Engineer Waterways Experiment Station (WES). Funds for the project were provided under Operations and Maintenance of the Army Program No. 4K07812AQ61 "Criteria for Tunnel and Cavity Support Systems." This report contains the results of two contract studies, DACA 39-74-M-0371 and DACA 39-75-M-0082, and an agreement with the U. S. Bureau of Reclamation. Contributors to this study were: Professor T. L. Brekke, University of California, Berkeley, California; Professor H. H. Einstein, Massachusetts Institute of Technology, Cambridge, Massachusetts; the U. S. Bureau of Reclamation, Engineering and Research Center, Denver, Colorado; and R. E. Mason, Senior Engineer, A. A. Mathews, Inc., Rockville, Maryland.

The studies were conducted during FY 74 and 75 under the direction of Messrs. J. P. Sale and R. G. Ahlvin, Chief and Assistant Chief, respectively, of the Soils and Pavements Laboratory. The contracts were monitored by Mr. J. S. Huie, Chief, Design Investigations Branch, under the general supervision of Mr. Don C. Banks, Chief, Engineering Geology and Rock Mechanics Division.

The Director of WES during these studies and preparation of the report was COL G. H. Hilt, CE. Technical Director was Mr. F. R. Brown.

CONTENTS

	<u>Page</u>
PREFACE	2
CONVERSION FACTORS, METRIC (SI) TO U. S. CUSTOMARY AND U. S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT	4
PART I: A DISCUSSION OF SOME AMERICAN DESIGN APPROACHES FOR UNDERGROUND OPENINGS IN ROCK	5
Introduction	5
Background	6
Shotcrete or Guniting Used as a Rock Sealant	8
Shotcrete Used as a Safety Measure	9
Shotcrete Used as a Structural Support	9
Conclusions and Recommendations	12
PART II: EUROPEAN SHOTCRETE DESIGN AND PRACTICE	14
Shotcrete Application in Europe	14
Shotcrete Mechanisms	16
Shotcrete Design	19
Shotcrete Technology	39
Developments Affecting Shotcrete Design and Practice	43
Recommended Research and Development in Both Design- Analysis and Technology	46
Summary and Conclusions	47
PART III: USE OF SHOTCRETE FOR TUNNEL LINING	49
Introduction	49
Design Considerations and Experiences	50
Construction Considerations and Experiences	66
Testing and Evaluating Considerations and Experiences	77
Summary and General Discussion	83
PART IV: SHOTCRETE INSTRUMENTATION	86
Introduction	86
Gloetzl Pressure Cells	88
Alternate Systems to the Gloetzl Cell	96
Borehole Extensometers	99
Recommendations for Shotcrete Research	112
Rational Approach to Tunnel Support Design	113
REFERENCES	116
BIBLIOGRAPHY	118

CONVERSION FACTORS, METRIC (SI) TO U. S. CUSTOMARY AND
U. S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

Units of measurement used in this report can be converted as follows:

<u>Multiply</u>	<u>By</u>	<u>To Obtain</u>
<u>Metric (SI) to U. S. Customary</u>		
millimetres	0.0394	inches
centimetres	0.3937	inches
metres	3.2808	feet
kilometres	0.6214	miles (U. S. statute)
square centimetres	0.1550	square inches
square metres	10.76	square feet
cubic metres	1.308	cubic yards
cubic decimetres	0.26417	gallons (U. S. liquid)
kilograms	2.205	pounds
metric tons	1.1	tons
kilograms per metre	0.6720	pounds per foot
megapascals	145.03894	pounds per square inch
kilograms per cubic metre (mass)	1.68557	pounds per cubic yard
Celsius degrees or Kelvins	9/5	Fahrenheit degrees*

<u>U. S. Customary to Metric (SI)</u>		
inches	2.54	centimetres
feet	0.3048	metres
miles (U. S. statute)	1.609344	kilometres
cubic yards	0.7645549	cubic metres
gallons (U. S. liquid)	3.785412	cubic decimetres
pounds (mass)	0.4535924	kilograms
pounds per square inch (force)	0.006894757	megapascals
pounds per cubic yard (mass)	0.59327631	kilograms per cubic metre
degrees (angle)	0.01745329	radians

* To obtain Fahrenheit (F) readings from Celsius (C) readings, use the following equation: $F = 9/5(C) + 32$. To obtain Fahrenheit from Kelvin (K), use: $F = 9/5(K - 273.15) + 32$.

STATE-OF-THE-ART REVIEW ON SHOTCRETE

PART I: A DISCUSSION OF SOME AMERICAN DESIGN APPROACHES FOR UNDERGROUND OPENINGS IN ROCK

by

T. L. Brekke*

Introduction

1. The purpose of this report is to provide an evaluation of the design techniques and the tunnel support philosophy of American governmental agencies and contractors, with particular attention to areas for future research. With this in mind, the report has been prepared as a discussion rather than as a standard review.

2. There is admittedly an additional reason for choosing this format: a survey of published literature, a study of available specifications, as well as numerous interviews with colleagues in the industry revealed that it is very difficult to identify a prevailing "American" approach in shotcrete design. When discussing design with these colleagues, it is indicative of the uncertainties involved that one very quickly is engaged in questions regarding wet versus dry mix, fluid versus dry accelerator, thickness per layer, amount of scaling, testing of shotcrete quality, etc. These are all directly pertinent questions in regard to performance, but only indirectly valid in terms of design.

3. The term "design" may in this context be defined as the integration of all factors believed to influence the behavior of a given underground opening where shotcrete will be used, followed by the development of specifics, such as minimum and average thickness, time of application in the excavation sequence, and definition of what constitutes an adequate job.

* Associate Professor of Geological Engineering, University of California, Berkeley.

4. It is therefore, for a planned opening, first necessary to identify those factors that will influence the outcome. Some of these have been listed below in the form of questions:

- a. What is the size, shape, function, and lifetime of the opening?
- b. What are the anticipated excavation methods and procedures?
- c. What are the geological characteristics of the site in terms of rock types? May they swell, slake, or dissolve?
- d. What major discontinuities, such as shears, seams, and faults, intersect the site? What are their spacing and geometry? What are the characteristics of the gouge material in terms of swellability, slakeability, erodability, or solubility?
- e. What are the geometry relative to the opening, spacing, roughness, shape, coating, and filling material characteristics of the detailed discontinuity pattern of joints and partings?
- f. What is the in situ stress picture at the site? Are the stresses so low that the fallout of rock blocks is greatly enhanced through the lack of confinement? Are they so high and/or anisotropic that rock burst phenomena may occur? Can squeezing be anticipated?
- g. What will the influence of the water regime be during and after construction?
- h. What measures in addition to shotcrete will be used to stabilize the opening?

5. It is most important to recognize that the interaction of all of these factors controls the behavior of the opening. Thus, it may be quite misleading to base the design on only a few of them, unless it has been assessed that the others have insignificant influence for a particular site.

Background

6. Shotcrete as defined by the American Concrete Institute (ACI 506-66) includes both pneumatically applied mortar and pneumatically applied concrete. In tunneling practice and literature, the term is most often used for pneumatically applied concrete only, and the term Guniting is used for pneumatically applied mortar. This distinction will be used here.

7. The first attempts to use Gunitite underground were made in Brucetown in 1914 by the Bureau of Mines on the initiative of Chief Engineer George Rice. Between the world wars, Gunitite was extensively used in American mines and was also introduced in other underground openings (e.g. the San Jacinto Tunnel in California).

8. After World War II, machines were developed in Europe for pneumatic application of concrete with aggregate up to 1 in.* in size. At the same time, the application of shotcrete close to the face as an integrated part of the driving process was initiated. This use of shotcrete, often in combination with rockbolts and wire mesh, rapidly spread through the Alpine countries and to the Scandinavian peninsula. Shotcrete and composite reinforcement/support systems that include shotcrete are today very widely used in underground openings in Europe and have proved to be both efficient and economical in stabilizing a number of types of adverse ground conditions. Shotcrete has also been used successfully in other parts of the world, including South America, Hong Kong, and Japan.

9. More recently, shotcrete has been used in Canada and in the United States. This late development seems to have several causes. Partly, it may be ascribed to some unsuccessful experiences with Gunitite. Used often as a rather thin overlay, it was sometimes ineffective, resulting in spalling and fallout. Shrinkage problems due to a high cement content would aggravate this situation.

10. A more important cause, however, for rockbolts as well as shotcrete, has been the almost total dependence on steel supports in civil engineering practice in North America. Long experience and tradition in the use of steel supports have very naturally led to some reluctance in accepting new and "lighter" methods even for ground conditions when they clearly would give sufficient reinforcement and be economically advantageous. Through Terzaghi's classical classification system and Proctor and White's design procedure (Reference 1), the design of

* A table of factors for converting metric (SI) units of measurement to U. S. customary units and U. S. customary units to metric (SI) is given on page 4.

steel supports was fully quantified in the sense that numerical values can be assessed for the rock loads, the load transfer between the supports and the surrounding rock mass analyzed in numerical terms, and the supports consequently designed.

11. Being used to this fully quantified design procedure, it is understandable that the tunnel designers have been reluctant in accepting new methods such as shotcrete without being provided with a well-established design procedure that includes relevant numerical terms for pertinent rock mass properties, gives a generally accepted method of analysis of rock/shotcrete interaction, and, finally, gives a clear procedure for finding the "amount" of shotcrete needed.

12. In the absence of a manual on shotcrete design, the designers have by and large based their design on conceptual considerations and on case histories with shotcrete as reflected in literature and in their own experience (if any). This has not taken place with ease--the question of how shotcrete acts as a rock mass stabilization measure is not satisfactorily answered as yet, at least not in numerical terms. In a sense, however, this is the wrong question. The right question to address is how a shotcreted rock mass acts.

Shotcrete or Guniting Used as a Rock Sealant

13. Thin applications of shotcrete or Guniting are sometimes used as a measure to prevent slaking of argillaceous rocks (e.g. shales) due to drying and/or wetting. They can be quite effective in protecting rock surfaces against these environmental changes, although they are perhaps at the present time not as economical and efficient as other sealants.

14. Used for this purpose, shotcrete and Guniting are not supposed to have a structural role; their purpose is only to prevent rock deterioration immediately adjacent to the opening. Thus, they will not be adequate measures if the slaking and sloughing are due to squeeze rather than environmental changes, or if high swelling pressures could develop.

Shotcrete Used as a Safety Measure

15. This heading covers the use of shotcrete to temporarily secure against localized small rockfalls that may pose a threat to personnel and/or equipment. Typically, this type of application is chosen by the contractor or specified spring-line to spring-line in large tunnels or other openings where the rock mass is inherently stable after proper scaling and, sometimes, reinforcement by rockbolts. The design considerations used for this particular application are purely empirical, and it is normal to use a rather nominal thickness.

Shotcrete Used as a Structural Support

16. This use of shotcrete may be divided into several groups:
- a. Unreinforced shotcrete, 2 to 4 in. thick.
 - b. Reinforced shotcrete.
 - c. Reinforced shotcrete used in conjunction with rockbolts or spiling reinforcement.
 - d. Shotcrete used alone or in conjunction with steel sets or rebar trusses in lieu of concrete lining.

17. The difference between these applications of shotcrete and the one mentioned in paragraph 15 is that they are used as a rock mass stabilization measure, and not just for safety against a localized fall-out that does not threaten the overall stability of the opening. To bracket the problem somewhat, the following comments are made with reference to blocky and seamy rock.

18. When an opening is excavated in such rock, elastic as well as permanent deformations will ensue. The deformations are time-dependent both as a function of the advancement of the excavation and as a function of the time-dependent characteristics of the rock mass as a material.

19. As the deformations occur, the inherent available rock mass strength is reduced due to strain softening and loss of confinement. In particular, this is critical immediately adjacent to the opening on account of the high friction angles associated with low confinement.

Beyond a certain amount of deformation, the rock arch may lose its continuity through the fallout of a "key" block, with ensuing raveling or slide.

20. Pursuing this concept, it seems clear that the most efficient way of stabilization of a blocky and seamy rock mass is to reduce the deformations and maintain rock arch continuity. It is relatively reasonable to envisage how rockbolts or spiling reinforcement can effectively achieve this. It is also easily understood that support in the form of steel sets or concrete lining can arrest the deformations, although less efficiently than rockbolts or spiling reinforcement.

21. It is harder to evaluate numerically how reasonably thin (2 to 4 in.) shotcrete can reduce the deformations and maintain rock arch continuity. Viewed as a thin arch subject to external loading, unreinforced shotcrete has a rather nominal capacity. Although this approach lends itself to analysis, there seems to be a general agreement that it is invalid.

22. The prevailing view is that 2- to 4-in. shotcrete cannot be considered "separately" from the rock mass, that it does not lend itself to analysis, but that it can be designed on the basis of experience. The structure that is formed is a composite rock mass/shotcrete structure. The essential effect of shotcrete used this way is to prevent "key" blocks from loosening and thereby causing loss of rock arch continuity.

23. There is ample evidence in literature as to the parameters that will influence the stability of such a structure. It is understood that these will be discussed in detail in the Shotcrete Manual that is presently being prepared at the University of Illinois (J. W. Mahar, personal communication).

24. The important factors as far as the shotcrete is concerned are the tensile strength and the shear strength locally at critical points, such as the protrusion of a rock block that may rotate out. Further, it seems that the shear strength of the bond between the shotcrete and the rock can strongly influence the outcome.

25. While it is readily understood that the tensile and shear strengths are important, it is presently less well established other

than in a qualitative way that the shotcrete/rock bond strength is of such significance as it appears to be. The most difficult bond problems exist when the rock blocks are defined by discontinuities with coatings of clay, chlorite, talc, serpentine, or graphite. It is in practice almost impossible to clean and prepare these surfaces to ensure a more adequate bond.

26. It is generally accepted that a reinforcement mesh can greatly improve the efficiency of shotcrete, and specifying mesh seems to be more and more common. It should be noted that several contractors are somewhat opposed to the use of mesh from a practical construction point of view. They would rather increase the thickness of the shotcrete. However, this may not at all be as efficient as a mesh under most failure conditions. The major objective of mesh reinforcement is to provide "post-failure" strengths.

27. The same, basically empirical approach seems to prevail for the design of composite systems involving rockbolts or spiling reinforcement and reinforced or unreinforced shotcrete. While most agencies will accept such a system for "temporary" (construction expedience) stabilization, some are still reluctant to accept them for permanent stabilization. This can lead to substantial overdesign with associated increase in cost.

28. Shotcrete has sometimes found use as a substitute for a conventionally placed concrete liner with a thickness of 6 in. or more. As such, it can and has been analyzed basically in the same conventional way as used for placed liners. Also, it has been used in combination with steel sets in two ways:

- a. Apply the shotcrete between the webs of traditionally blocked sets some distance behind the face to provide rigidity and to improve thrust capacity of the composite support system. This system may provide the final lining if the sets are covered by shotcrete and are consequently protected against corrosion.
- b. Apply shotcrete at the face to act as a continuous blocking and lagging of the sets in addition to its normal function. This method has, for example, proved to be successful in some underground openings that are part of the Washington, D. C., Metro.

29. A substantial reduction in the amount of steel can be made when shotcrete is used as a continuous blocking/lagging. The moment capacity is considerably higher for heavier, traditionally blocked steel with the same thrust capacity. However, with the continuous "blocking" provided by the shotcrete, the moments that can develop are considerably smaller than those that can be found with traditionally spaced blocking points.

30. The overbreak that most often ensues from drilling and blasting will generally lead to a considerable increase in the volume of shotcrete used if it is specified that the shotcrete between the webs has to be within a certain distance from the inner flanges of the sets. It should be recognized that the capacity of the composite system is not significantly changed even if the inside line of the shotcrete is moved somewhat further back. Alternatively, rock protruding between the sets can well be allowed as the shotcrete/rock structure possesses a considerable load-carrying capacity.

31. An important practical aspect is that the capacity of present shotcreting machines is rather limited. Although several machines may be used simultaneously, in large openings with substantial quantities required over short periods of time, pumped-in-place concrete at the face may be more economical.

Conclusions and Recommendations

32. Based on this discussion, the following conclusions and recommendations are made:

- a. The fundamental objective at the design stage of an underground opening is to predict ground behavior and then ascribe the measures that will most efficiently stabilize the opening.
- b. The ground behavior will depend on a number of factors as explored in Part I. It is inherently nonlinear and time-dependent.
- c. Except when used as a concrete liner, there seems to be concurrence that the design of shotcrete as structural support does not lend itself to analysis. However, most colleagues believe that it is possible to empirically design the structure involving shotcrete on the basis of

experience accumulated here and abroad. In fact, the prevailing view is that it should be based on experience. Of significant help, therefore, would be an inventory on case histories that in detail describes and evaluates the factors mentioned in Part I, with particular attention to failures. This is a rather formidable but most important task.

- d. The use of numerical techniques, such as the finite element method, could be helpful in considering specific design options when and if constitutive models based upon in situ experimental data for a given site could be identified. Failure data would be an essential part of the in situ test methods. It does not seem possible, or even appropriate, to develop design codes that are based on laboratory-measured properties only.
- e. Physical models, coupled with numerical models, could enhance the understanding of the basic behavior characteristics of a shotcrete/rock structure.
- f. The importance and effect of the shotcrete/rock bond strength should be studied experimentally.

PART II: EUROPEAN SHOTCRETE DESIGN AND PRACTICE

by

H. H. Einstein*

Shotcrete Application in Europe

Historic development

33. The development of shotcrete actually started with the United States invention of the mortar gun and Gunitite in 1907. The main purpose of guniting was and still is the mechanized application of mortar (Gunitite). The idea was quickly taken over by the Europeans, and already by 1911 it was successfully used in a tunnel as an overlay of the badly deteriorated lining. Similar applications of Gunitite for protection of liners or rock surfaces continued in particular to prevent water seepage and ice development in tunnels. After the invention and development of the shotcrete gun by the Swiss firm Aliva, shotcrete was first used in tunnels at the Maggia hydro-development. The reason for using shotcrete was that the mechanized application was faster and saved formwork. In free flow tunnels, the increased roughness could be easily made up by a somewhat larger cross section that together with the shorter construction time resulted in substantial savings. It was only 3 yr later that Sonderegger (Reference 2) pointed out the structural advantages of a temporary support that can be quickly applied after excavation and that is inherently flexible. This idea was further developed by Rabcewicz (Reference 3) and led to the New Austrian Tunnel Method (NATM). This short review of the development pointed out some of the applications that shall now be described in a more formal manner.

Shotcrete applications

34. It must be remembered that this description has as its goal to provide an overview. The detailed aspects of the applications will be discussed later.

* Associate Professor of Civil Engineering, Massachusetts Institute of Technology, Cambridge.

35. Tunnel applications.

- a. Structural support. The temporary shotcrete support will be incorporated in the final support, for example, cast-in-place concrete. Shotcrete in this application may be used alone, together with bolts or steel sets, or both. It can be reinforced or consist of plain concrete. Shotcrete may also be used to cover only the crown, the crown and various portions of the sidewall, or the entire cross section.
- b. Prevention of alterations. A protective coat of the natural rock is frequently necessary to prevent slaking and sloughing due to exposure to air, other gases, and water; to prevent frost weathering; to protect the rock or existing liner from mechanical wear due, for example, to flowing water. The protective coat can be either Gunitite (mortar) or shotcrete.
- c. Prevention of seepage. Gunitite or shotcrete has been extensively used to control "area seepage" (the flow of water from extended surfaces, in contrast to the preferred flow from concentrated channels) by applying an impermeable coat and providing relief holes. Also in the last few years, concentrated flow is being handled by shotcreting surfaces together with preinstalled channels and relief pipes. An important related area is the prevention of ice formation (ice can form either where the water emerges or where it falls; the latter causes severe problems in cases of electric railroad installations and road surfaces).
- d. Support layer for water stop. The recent trend in tunnel design and construction is to include water stops in the liner of all major tunnels. These water stops consist of sprayed or rolled-on liquids that solidify in place or of sealing sheets. Neither type can be applied to the natural ground surface. Shotcrete is used to equalize the major surface irregularities, and frequently a Gunitite layer is used in addition to provide a smooth surface.
- e. Drainage layers. Special coarse aggregate shotcrete is used to provide a drainage layer under regular shotcrete or another kind of liner.
- f. Repair and reconstruction. Any of the aforementioned applications can be used in the context of tunnel repair or reconstruction. The European railroads are involved at the present time in a tunnel reconstruction program of considerable importance where shotcrete is extensively used.

36. Nontunnel applications. Naturally, any of the purposes

mentioned in the context of tunneling (structural, prevention of alteration and seepage, support for water stop, and drainage) apply here also. The main reason why shotcrete is used in nontunnel applications is the elimination or reduction of formwork. Some of the more important areas of application shall be mentioned here.

- a. Open "tub" or "dish-shaped" structures in the ground. Such structures frequently have relatively steep sides where counterforms or small concreting stages would be necessary. The use of shotcrete simplifies this operation significantly. Shotcrete can be reinforced or plain. It can be applied directly against the ground or against a drainage layer. Many large-diameter tanks in sewage treatment plants are constructed in this manner.
- b. Slope protection. The temporary or permanent protection of cut slopes mainly in connection with road construction is another major field of shotcrete application. The main purposes are the prevention of alteration and of seepage (sometimes with a separate drainage layer).
- c. Shotcrete structures. In cases where architectural design involves complex surfaces, shotcrete saves considerably on formwork. Architects sometimes also require the use of shotcrete to give a sculptural appearance to the structure.

37. The various applications of shotcrete can be associated with one of two major physical purposes: (a) structural purposes (tunnel support, support for water stop, open structures in ground, shotcrete structures); and (b) sealing and drainage (prevention of alteration and seepage in tunnels, slope protection, drainage layers). The following paragraphs will review the mechanisms that contribute to these purposes.

Shotcrete Mechanisms

Shotcrete used for structural purposes

38. Shotcrete alone reacts in the same manner as any reinforced or plain concrete by supporting stresses due to bending, thrust, and shear. The differences in material technology--relatively lower strength and lower unit weight compared to regular concrete--have no effect on the basic mechanism. The usually faster setting time and strength increase

of shotcrete allow application to steep and vertical faces and, when accelerators are used, also to overhang faces without counterforms. In such applications, it is likely that the shotcrete adheres to some extent to the face (existing structure, form panel) and to reinforcing steel, a mechanism which involves a combination of shear and tensile stresses.

39. Shotcrete interacting with the ground involves several mechanisms that are uncommon in standard structural concrete:

- a. The shotcrete adheres to the ground by a combination of shear and tensile stresses between the ground and the shotcrete and within the shotcrete (analogous to the second mechanism mentioned above).
- b. The shotcrete fills openings (fractures, joints) in the ground adjacent to the application surface. This permits a transmission of compressive, shear, and tensile stresses in this zone of the ground.
- c. The two characteristics--adherence to the ground and filling of fractures (openings)--lead to a tightly interacting ground-shotcrete structure. Due to the adherence and interlocking by reaching into fractures, it is possible to transmit shear stresses with no or little slippage between shotcrete and ground and also to transmit tensile stresses. The shotcrete and adjacent ground behave thus, to quite an extent, as a composite structure. (It should be pointed out that although this ground-shotcrete characteristic is particularly important in rock, it also applies, to a certain extent, to soft ground due to the fact that the shotcrete impact may lead to a degree of interlocking.) The ground-shotcrete interaction is not only important with regard to the use of shotcrete as a support but also as protection against mechanical wear, e.g. in water tunnels.
- d. It occurs frequently that parts of the ground (individual rock blocks, plastic zones in the ground) move or deform and shear through the shotcrete. This shearing mechanism is the same that any normal concrete structural member may undergo; it is mentioned specifically due to its importance in tunnel support design.
- e. One of the most important shotcrete characteristics is the flexibility, particularly of thin shotcrete layers used for tunnel support. The flexibility is not only due to the use of thin layers but also due to the fact that the shotcrete acts structurally immediately after setting. In other words, the viscous behavior of the "green" shotcrete under load is deliberately used. The basic mechanisms (bending, thrust, shear) do not change, but due to

the flexibility bending becomes relatively unimportant compared to thrust and shear.

- f. The usually uneven ground surface, particularly in a blasted tunnel, leads to an equally uneven shotcrete layer. This increases the moment of inertia of the shotcrete layer if it is considered independently of the ground. It increases the aforementioned interlocking effect. Also, the effective thickness of the shotcrete increases due to the inclined surfaces. The effect of the protruding edges and corners is somewhat disputed. Mahar, Gaw, and Cording (Reference 4) state that these are the locations of the first fractures because of tensile stress concentration and usually thinner shotcrete. Several European engineers, however, make the point that the protruding surfaces in a blasted tunnel are more resistant to start with and therefore need less support.

It should be pointed out here that many if not most of these characteristics and mechanisms are uniquely related to shotcrete. Cast-in-place concrete and precast concrete cannot enter fractures to the same extent, do not follow irregularities and adhere as well, and particularly cannot be constructed in thin layers and are thus more rigid.

Shotcrete for sealing and drainage

40. Sealing is mainly achieved by the inherent material technology of shotcrete, i.e., high cement content, relatively low water-cement factor, and large volume of closed pores.

41. Prevention of water flow. The high degree of impermeability prevents or limits flow of water from the ground to the atmosphere, and vice versa. The reduction of water flow into the ground reduces adsorption and absorption and, thus, also the associated mechanisms of slaking, swelling, and mineral alterations. Also, the frost resistance of the ground will be increased. The reduction of flow from the ground prevents erosion of joint fillers and fault gouge and thus contributes to the stability of the ground, in addition to a reduction of adverse effects caused by the water itself (see paragraph 35). However, the prevention or reduction of water flow from the ground may also lead to a buildup of seepage stresses and possibly to strength decrease and stability problems in the ground.

42. Reduction of gas flow. The high degree of impermeability

reduces also the flow of air and other gases through the shotcrete. Thus, the complete drying out of the ground, which would lead to substantial shrinkage, is prevented. Chemically aggressive gases are impeded in their flow. The cement-rich shotcrete itself is also relatively more resistant than normal concrete. Both factors have the effect of a protective coating.

43. Drainage layer. A highly permeable but well-cemented coarse aggregate shotcrete permits flow without being eroded. Permeabilities in the range of gravels can be expected.

Shotcrete Design

44. The preceding review of shotcrete mechanisms provides the basis for the following discussion of design approaches which have to take these mechanisms into account.

45. The discussion of European shotcrete design will follow the same sequence as the discussion of shotcrete applications (see paragraphs 34-37):

- a. Structural shotcrete in tunnels, i.e. structural tunnel supports.
- b. Nonstructural shotcrete in tunnels.
- c. Other shotcrete applications.

The emphasis will be on the first category.

Design of shotcrete tunnel supports

46. The design of most tunnel supports consists of two design processes: the design of the ground-support system and the detailed design of the support. The following discussion will be structured correspondingly and will also include the design of other types of tunnel supports.

47. Design of ground-support systems. The modern ground-support systems, particularly those involving shotcrete, take the basic ground-support interaction mechanism into account. This mechanism is a transition from the

primary state of stress	(natural state of stress)
to the	
secondary state of stress	(immediately after creating the
to the	opening)
tertiary state of stress	(ground interacting with the
	support)

Both the secondary and tertiary states of stress depend on time and on the location of the point in question relative to tunnel advance and support installation. The stresses in both states are thus not constant. With this principle established, the major design approaches are as follows:

- a. Empirical approach. Linder (Reference 5) presents a modified version of the Lauffer Rock Classification diagram. (This diagram is reproduced in Deere et al. (Reference 6) and will therefore not be presented here.) The Lauffer diagram relates rock class, opening width, standup time, and timber support requirements based on observations. Linder uses the same rock classes but assigns modern supports, i.e., nonreinforced or reinforced shotcrete in various thicknesses and in various combinations with steel sets and rockbolts. The problematic aspect of this approach is the determination of rock classes, which is mainly based on personal experience. If used by experienced engineers, it is a valuable design tool.
- b. Characteristic curve approach. The characteristic curve is a design tool that was used extensively around the turn of the century and has been recently rediscovered. As shown in Figure 1, it is a stress-displacement curve of a specific point at the tunnel circumference or of an "average point" at the circumference relating a virtual

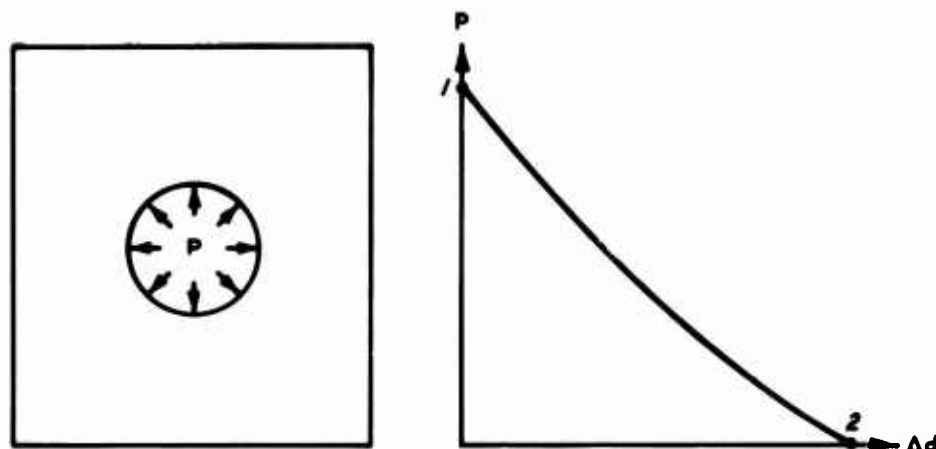


Figure 1. Characteristic curve for a tunnel

internal pressure to the displacement of the circumference. Peck (Reference 7) uses this approach also. Point 1 (Δd (displacement) = 0, p = original or primary stress) represents the original state or primary state; somewhere between 1 and 2 the curve represents the secondary state. Lombardi (Reference 8) uses an approximation to determine this secondary state (Figure 2). The characteristic curve

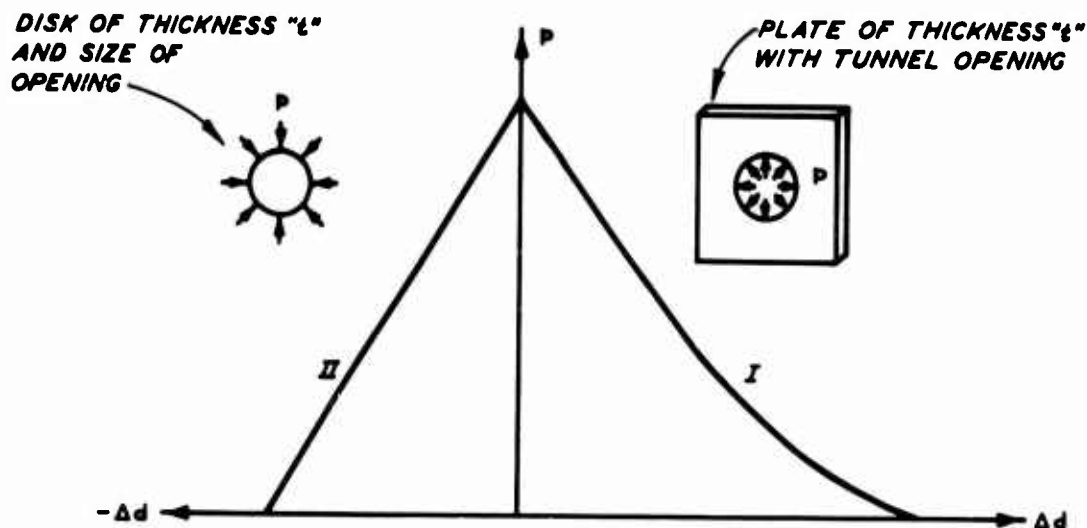


Figure 2. Characteristic curve for a tunnel and disk

is derived by assuming a plate of finite thickness t which is subject to the in situ state of stress and in which a hole is created by reducing p from the primary value to 0 (curve I). A disk of the same size as the original opening (i.e. at $\Delta d = 0$) is also subject to p being reduced from the original value to 0 and results, for example, in characteristic curve II. The thickness of the disk is now reduced to a fraction of t (Figure 3), and the disk is virtually reinserted into the opening, resulting in curve IIIa assuming elastic behavior and curve IIIb assuming elastoplastic behavior. Points F in Figure 3 represent the state of stress at the face, i.e. the secondary state of stress. The question now arises which fraction of t has to be chosen to represent the actual behavior in a tunnel. Comparison between computed characteristic curves and observations in tunnels has shown that $t/2$ is a reasonable value. (This may, however, change if more observations become available, and it may turn out to be material and opening-size dependent.) The characteristic curves can be obtained by the finite element (FE) method for continua or discontinua, elastic or elastoplastic behavior. Lombardi (Reference 8) uses a numerical (computer-based) but somewhat simplified approach, which starts with an estimated

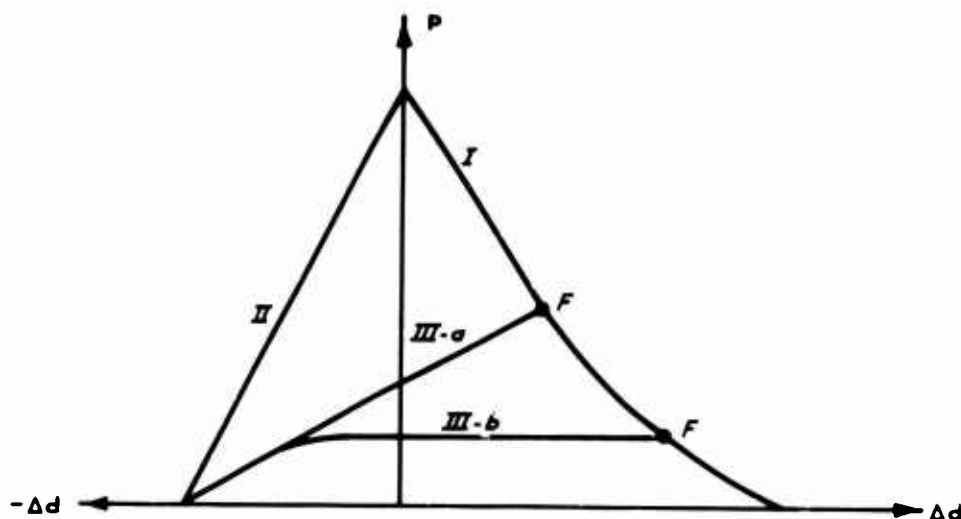


Figure 3. Characteristic curve including effect of the tunnel face

location of the elastoplastic interface, for a certain value of p in the ground; then he computes stresses attempting to satisfy equilibrium, compatibility, and boundary conditions in the elastic zone and the yield criterion, equilibrium, and boundary conditions in the plastic zone. The conditions are satisfied by shifting the elastoplastic interface in an iterative procedure. This procedure can be used for a continuum or a discontinuum; the latter by making the yield criterion dependent on the direction of the discontinuities. Based on the stress state, displacements are computed in a quasi-elastic procedure. The dilatant behavior of the ground is taken into account by adding a volume increase in the plastic range (Figure 4). The values of ΔV vary between 0.1 and 0.5 percent for most rocks. As shown in a recent paper by Lombardi (Reference 9), Figure 5 illustrates that it is also possible to take time effects into account. However, only a limited amount of observations are available for comparison with this analytical method. The characteristic curves lend themselves very well to the representation of ground-structure interaction. As shown in Figure 6, characteristic curves can be produced for any kind of tunnel support. The ground and support curves can be combined. As shown in Figure 7, the tunnel circumference displaces from l to F in the transition from the primary to the secondary state of stress. Some distance behind the face, a flexible (e.g. shotcrete) support is installed (curve S). Point I represents the state of equilibrium support ground at t_2 , i.e., when support and ground interact. The high degree of flexibility leads to relatively low stresses p_s acting on the

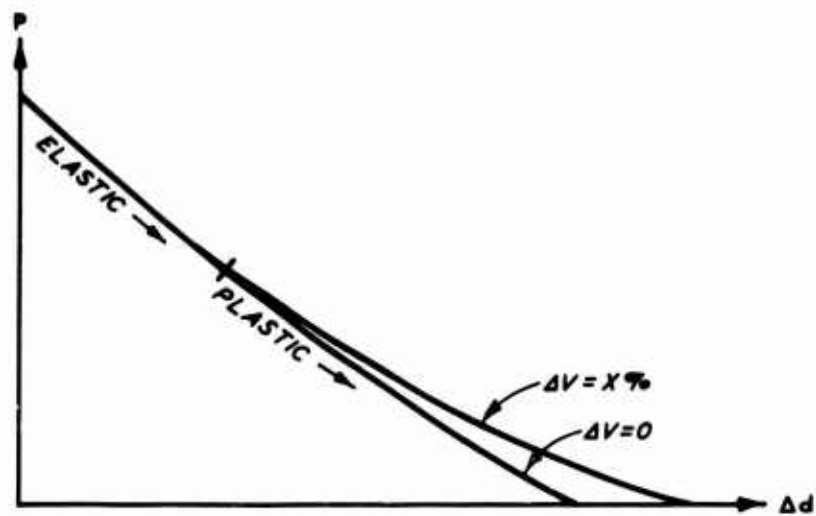


Figure 4. Characteristic curves with plastic volume increase

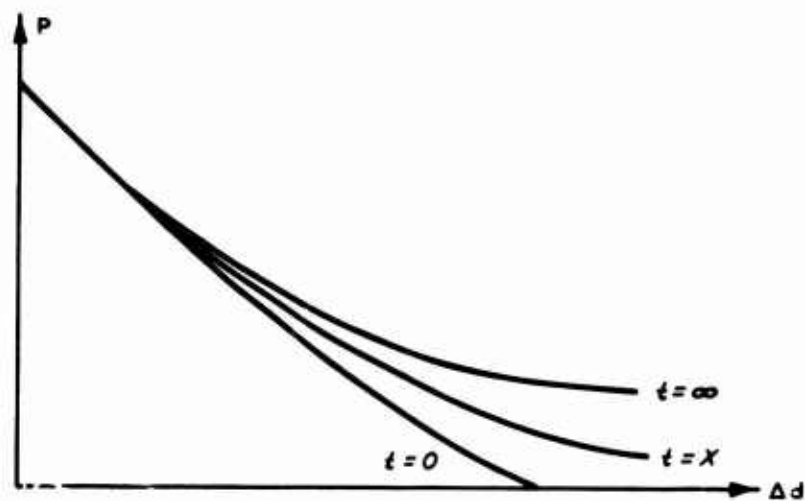


Figure 5. Characteristic curves with time effect

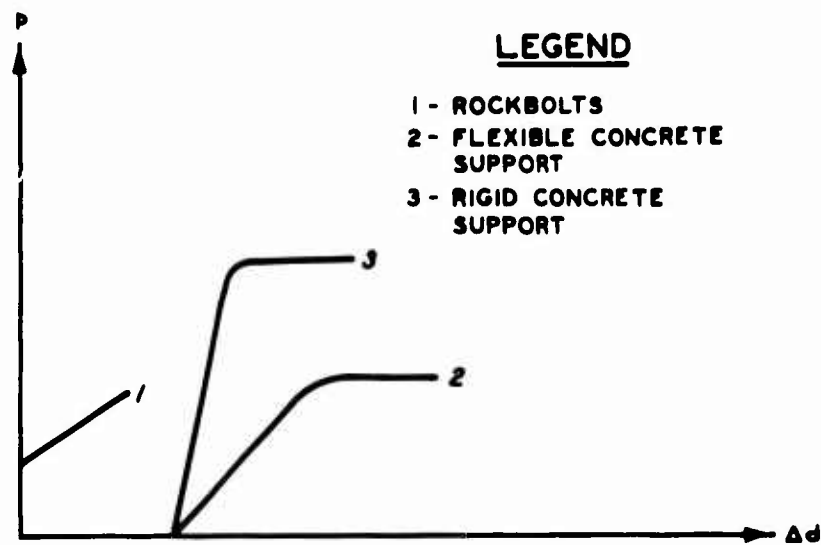


Figure 6. Characteristic curves of different tunnel supports

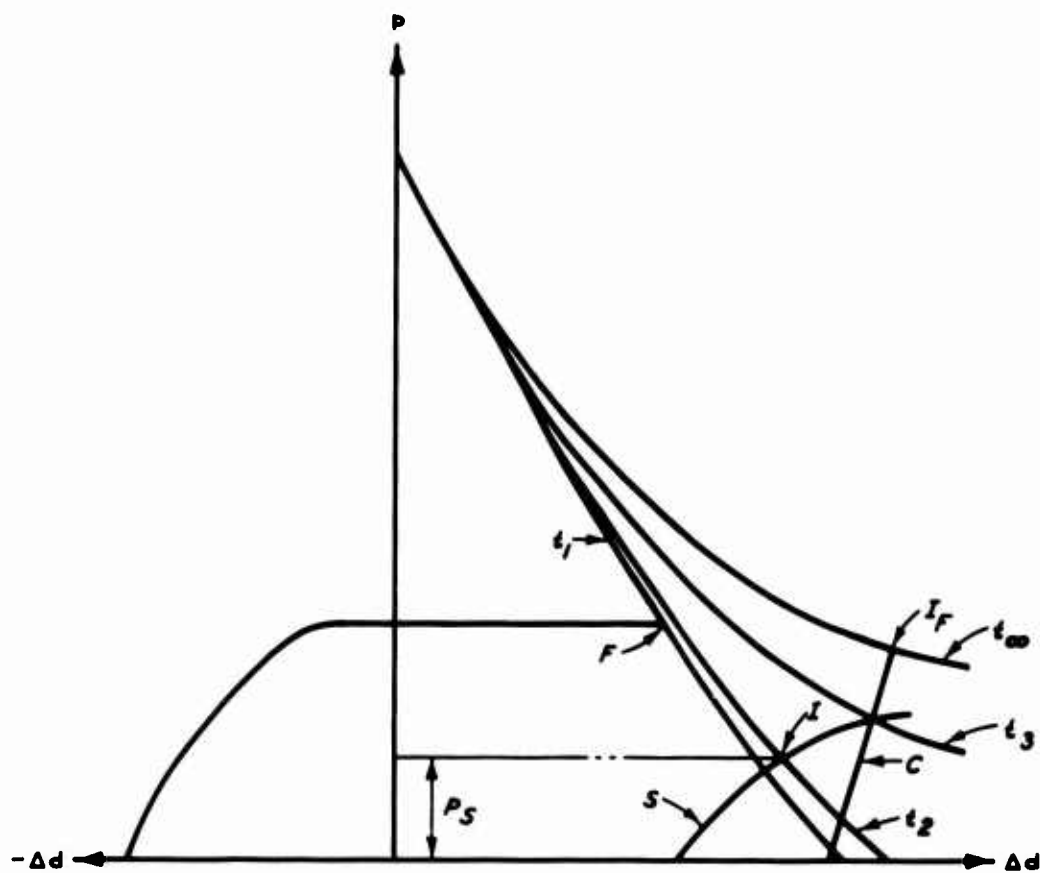


Figure 7. Characteristic curves showing ground-structure interaction

support, however, with relatively large displacements. The shotcrete support would lead to unacceptably large long-term displacements (intersection of curve S with curve t_{∞}). Thus, a secondary stiff concrete liner is installed (curve C) interacting with the ground at t_3 and reaching the final state represented by I_F . The combination of ground and support behavior can thus be represented by the characteristic curve approach. Although not geared toward shotcrete in particular, it permits the designer to examine the feasibility of shotcrete support (permissible displacements), and to determine the time when shotcrete should be applied and the combination of shotcrete with other supports, either simultaneously or subsequently installed. It should be pointed out, however, that the characteristic curves of the ground, particularly the volume increase and time dependency features, depend to quite an extent on experience. They can be incorporated relatively easily into Lombardi's numerical approach or in a FE analysis. The appropriate input variables have to be obtained from a set of past observations, which are only available to designers that have been involved in several tunnel projects.

- c. Combined static equilibrium--stress deformation approach. Two analyses are conducted independently: an analysis considering the equilibrium of stresses around the tunnel, and a stress-deformation analysis resulting in the characteristic curve. The approach is, therefore, very similar to the characteristic curve approach, which also considers stress equilibrium and stress-deformation behavior separately. However, some of the details are different, particularly the fact that this approach is simpler and can be performed without a computer.

(1) Stress equilibrium

- (a) Primary state of stress. Measured or approximated by assuming $\sigma_v = \gamma h$ and $\sigma_H = \gamma \cdot h \cdot K$. Usually σ_v and σ_H are principal stresses except near slopes. In the latter case, computation of magnitude and direction of the principal stresses is necessary.
- (b) Secondary state of stress. An elastic analysis based either on closed form solutions or on the FE method is performed to arrive at tangential and radial stresses. In case of rock with pronounced horizontal or near horizontal bedding, foliation, or jointing, the crown section of the tunnel is approximately modeled as a laminated beam with no shear stress transmitted between

laminae and analyzed elastically. The resulting tensile stress can be reduced up to 50 percent depending on the actually existing shear stress transfer. If the strength of the material is exceeded, a plastic analysis will be performed using closed form approaches for simplified conditions (uniform stresses), e.g. Obert and Duvall (Reference 10) and Kastner (Reference 11), or elasto-plastic finite element analyses. The main purpose is the determination of the boundary of the plastic zone, the stresses there, and the stresses at the tunnel circumference.

(c) Tertiary state of stress. The principle of the change from the secondary to the tertiary state of stress can be represented in a Mohr diagram (Figure 8) for the stress state at the elasto-plastic boundary, assuming σ_r = radial stress = minor principal stress; σ_θ = tangential stress = major principal stress. Cases I, II, and III can be distinguished:

- I. σ_θ remains constant; σ_r increases ~ stiff support.
- II. σ_r remains constant; σ_θ decreases ~ ideally flexible support (stiffness ~ 0).
- III. σ_r increases; σ_θ decreases ~ flexible support.

Case III corresponds to most modern supports, in particular to shotcrete. The increase of the radial stress σ_{r3} (III) - σ_{r2} = p_e . As shown in Figure 9, p_e can be assumed to act at the elastoplastic boundary. The stress p_i acting on the tunnel circumference is also the stress on the support and depends directly on p_e . Thus, it is evident that the stiffer the support, the greater p_e and the greater p_i . The stress p_i can be related to p_e , using the theory of the thick walled tube

$$p_i \left(\frac{\sigma_{\theta 3}}{p_i} + \frac{a^2 + 1}{a^2 - 1} \right) = p_e \frac{2a^2}{a^2 - 1}$$

where

$$a = r_p / r$$

r_p = radius of elastoplastic boundary

r = radius of tunnel

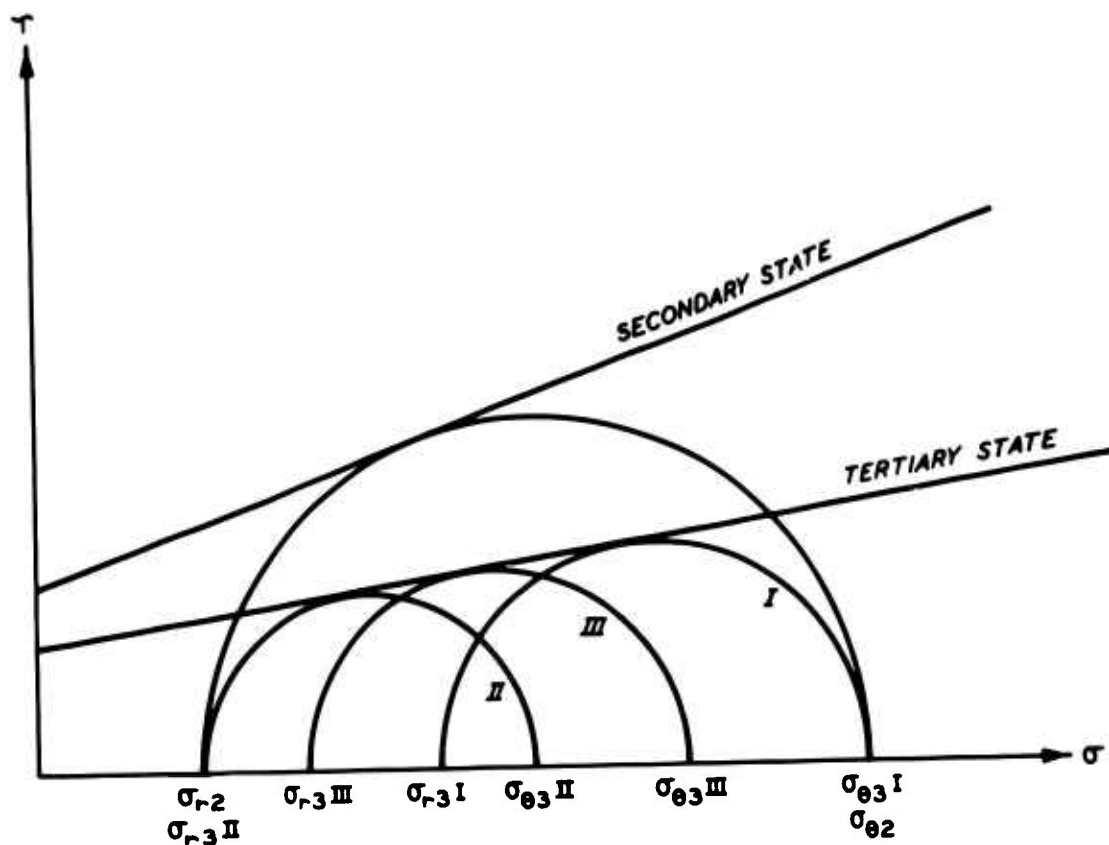


Figure 8. Mohr diagram for the secondary and three cases of tertiary state of stress

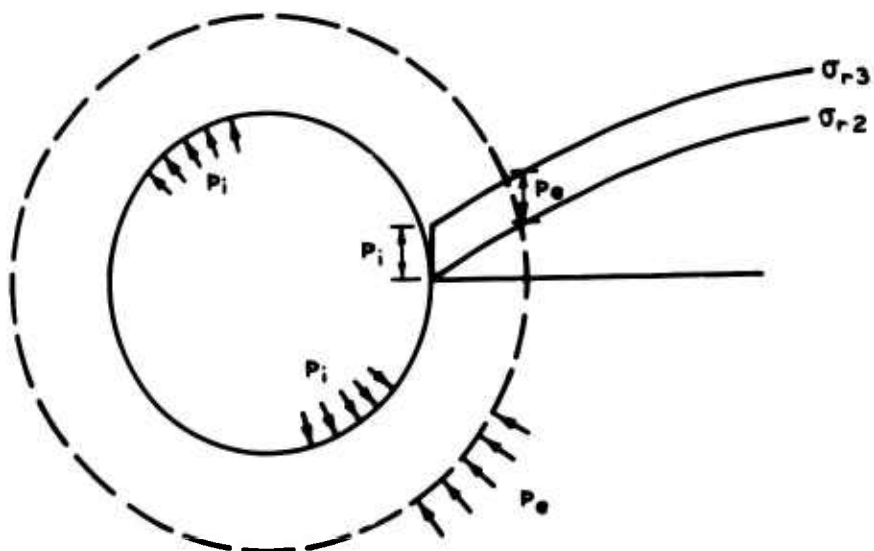


Figure 9. Stresses p_i and p_e acting on the plastic zone

An iterative procedure has to be used since $\sigma_{\theta 3}$ depends also on p_e (see Mohr diagram in Figure 8). Furthermore, p_e has to be estimated; its magnitude depends on the rheologic behavior. There are some empirical values correlating p_e with the maximum stress increase in the ground, which is usually the difference, $\sigma_{\theta 2} - \sigma_{I1}$ (major principal stress in secondary state of stress - major principal stress in primary state of stress), but the data base is small.

(2) Stress-deformation analysis. Characteristic curves are computed using the above determined stresses and stress-strain-time properties of the ground and the support.

(a) Primary to secondary state of stress. Displacements in the elastic zone are computed using closed form or FE analyses. Plastic displacements are normally computed using empirical values for volume increase ϵ_v . As observations have shown, ϵ_v for rock varies between 0.1 and 5.0 percent, depending on rock characteristics. (Note: This volume increase is not the same as the ΔV shown in Figure 4; ϵ_v includes the entire volume increase for elastoplastic deformation in the plastic zone; and ΔV represents only the volume increase in addition to the displacement computed by a simplified quasi-elastic analysis.) The displacement at the tunnel circumference (Figure 10) is computed by assuming that the volume increase ϵ_v of the plastic zone will result in uniform displacement toward the interior of the tunnel.

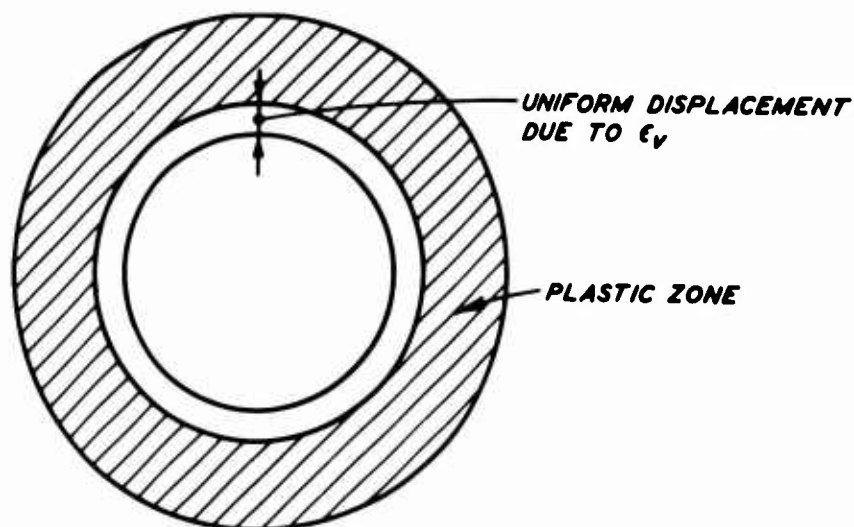
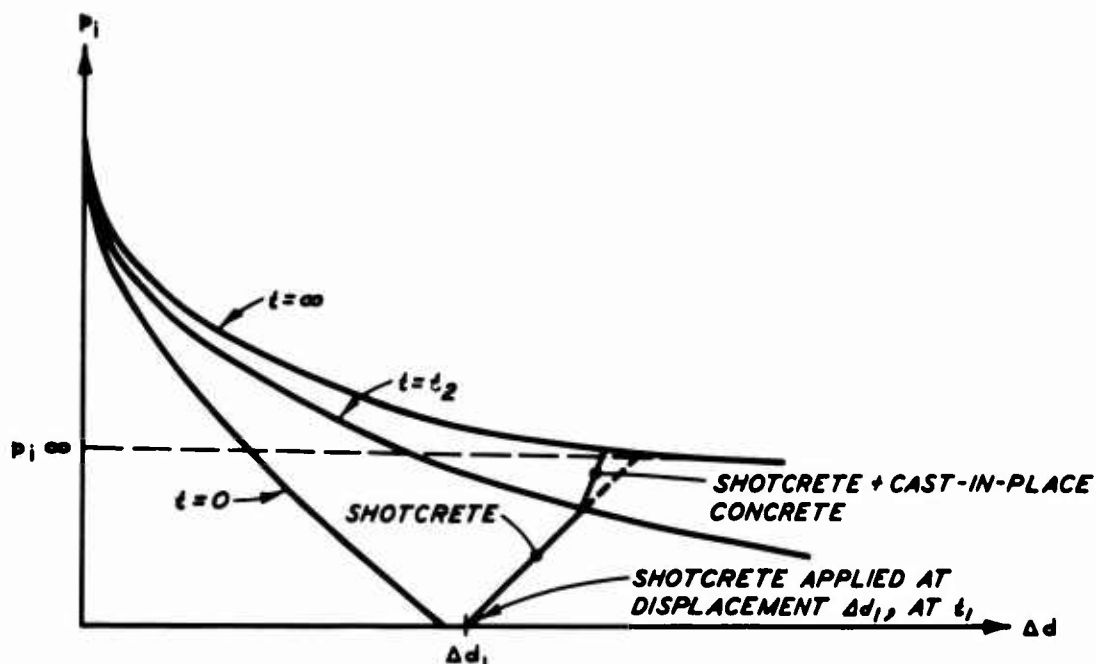


Figure 10. Displacements due to volume increase

(b) Secondary to tertiary state of stress. This step, illustrated in Figure 12, is analogous to the procedure of the characteristic curve approach: construction of additional time-dependent characteristic curves; construction of the characteristic curve for the support; and intersecting appropriate curves.



29

The time-dependent characteristic curves are again based on empirical data, which are limited at the present time. The resulting value p_i should approximately correspond to p_i as obtained by the stress equilibrium approach. It is evident that the combined "static equilibrium-stress deformation approach" and the characteristic curve approach follow the same principles. The characteristic curve approach is much more sophisticated. On the other hand, the "combined" approach does follow to some extent more traditional engineering analysis and may therefore give the user a better feeling for what he is doing. It should be emphasized, however, that the ground-structure interaction and the time effect, which are particularly important in shotcrete design, can be well expressed with both methods.

- d. NATM. This method, which is usually associated with shotcrete support in tunnels, is actually more than a design method. It includes design, construction, and performance monitoring in an integrated manner. It should also be pointed out that it is not restricted to shotcrete, as will be shown below, and that the basic concept was originally developed for bolt support (Reference 3). The principle of the NATM is the mobilization of the maximum possible support action by the ground itself. First, the development of a plastic zone around the tunnel is intentionally tolerated in order to reduce the peak stresses and to shift the location of the peak stresses away from the tunnel. Second, the deformations, however, must not reach a material-dependent magnitude that would reduce the resistance of the ground significantly, particularly in the zone near the tunnel surface. Rabcewicz calls this second mechanism "prevention of loosening." Muller, as cited by Louis (Reference 12), observes that a deformation of 2 percent may lead to a 90 percent reduction of shearing resistance. Rabcewicz (Reference 13) states that mechanism 1 (development of the plastic zone) takes longer than mechanism 2 (reduction of resistance), the latter taking place "almost immediately" after excavation. A progressive stress-deformation mechanism seems thus to govern the behavior in the ground around a tunnel. The tunnel support has thus to possess the following characteristics: be flexible enough to allow the development of the plastic zone (in other words, be flexible over the time period necessary for this development); and provide sufficient counterstresses at the tunnel-ground interface to increase the resistance of the ground

near this interface in order to prevent loosening (displacements that would lead to a significant loss of ground resistance).

- (1) The flexibility requirement can be satisfied with shotcrete of a thickness in the range of maximum one-fortieth to one-fiftieth of the tunnel diameter, probably in combination with bolts and light steel sets (allowing bolts and steel sets to deform up to and beyond general or local yielding). The fact that the shotcrete strength is relatively low at the beginning and increases with time is particularly advantageous and consciously taken into account.
- (2) The counterstress or resistance requirement can be satisfied by the shotcrete support again in possible combination with both steel sets and bolts. The shotcrete is particularly well-suited due to the fact that it provides a continuous support of the ground and that it can be applied very rapidly and, if necessary, immediately after excavation. The mechanisms described in paragraphs 39-45 provide the increase in resistance of the shotcrete-ground zone, and the deformation of the shotcrete leads to counterstresses. The counterstresses that can be provided depend significantly on the portion of the cross section covered by shotcrete. A full ring produces more counterstresses than an arch. In some cases, it may also be necessary to shotcrete the face, to provide counterstresses there, and to ensure its stability. In cases where the required shotcrete dimensions would exceed the aforementioned limits and would thus lead to a rigid behavior, it is imperative to provide the necessary counterstresses and resistance increase with bolts or light steel sets, which are again flexible supports. Both can contribute to the counterstresses and the resistance increase; the latter by "reinforcing the ground."

Observations show that, due to the flexibility of the support structure, failure occurs by shear in the shotcrete. Shear fractures are inconsequential as long as no excessive displacements, hence no loosening, take place, particularly since the shotcrete is considered a temporary support only. If the displacements become excessive, bolts and steel sets may have to be added. In many cases, it may be sufficient to simply spray shotcrete over the fractured parts. This conceptual description contains many qualitative terms and thereby reveals the main characteristics of the NATM--the great importance of empirical input in form of observations and experience. This will

become more evident in the following outline of the design procedure.

(1) Preliminary design of support system (Reference 14).

A characteristic curve without time effect is produced analogous to the methods described above. Based on experience, a displacement value is defined at which loosening starts; a corresponding stress p_i minimum (Figure 13) can then be obtained from the characteristic curve. The counterstresses of the support structure have to be greater than p_i minimum. As shown in Figure 14, the required support stresses are obtained through a force equilibrium analysis on the Rabcewicz shear body. The resisting forces are transformed into an equivalent horizontal stress p_{iw} acting over h . By fulfilling the requirement $p_{iw} \geq p_i$ minimum, it is possible to obtain the dimensions of the support (Reference 14) in an iterative procedure. The following empirical indications may be of use in the first step of this iterative procedure:

p_i minimum for 10-m-diam tunnels is usually in the range 30-90 kg/cm².

Approximate shotcrete thickness $d = 1/40 \div 1/50$ of the diameter for tunnels in the 5- to 10-m-diam category; and $d = 0.017 \times \text{radius}$ (Rabcewicz) for 3- to 5-m-diam tunnels.

For 1-m round length and 3- to 4-hr standup time, use bolts only; for 2-hr standup time, use steel sets; for shorter standup time, use shotcrete.

Bolts are 3 to 6 m long; bolt force is 20 tons (metric); density ranges from 1 per 4 m² to 1 per 20 m².

Based on the preliminary design, support requirements (dimensions and type) are determined for a number of typical "rock classes" in a particular tunnel.

(2) Final design and construction. The preliminary design dimensions are given to the contractor. Determination of the rock class is made on site by the contractor's representative and the resident engineer, and the contractor follows the corresponding construction procedure (round length, support dimensions, and sequence of support installation). Most importantly, extensive instrumentation is installed and observed. Simple fix points for diameter measurements (usually measured in two orthogonal directions) are installed every 15 m; extensometers are installed approximately every 30 m.

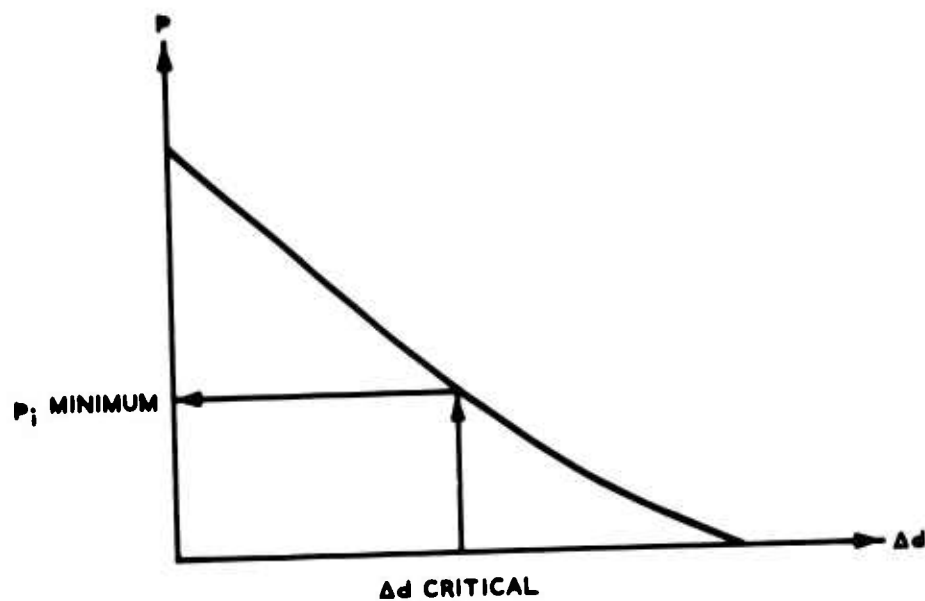
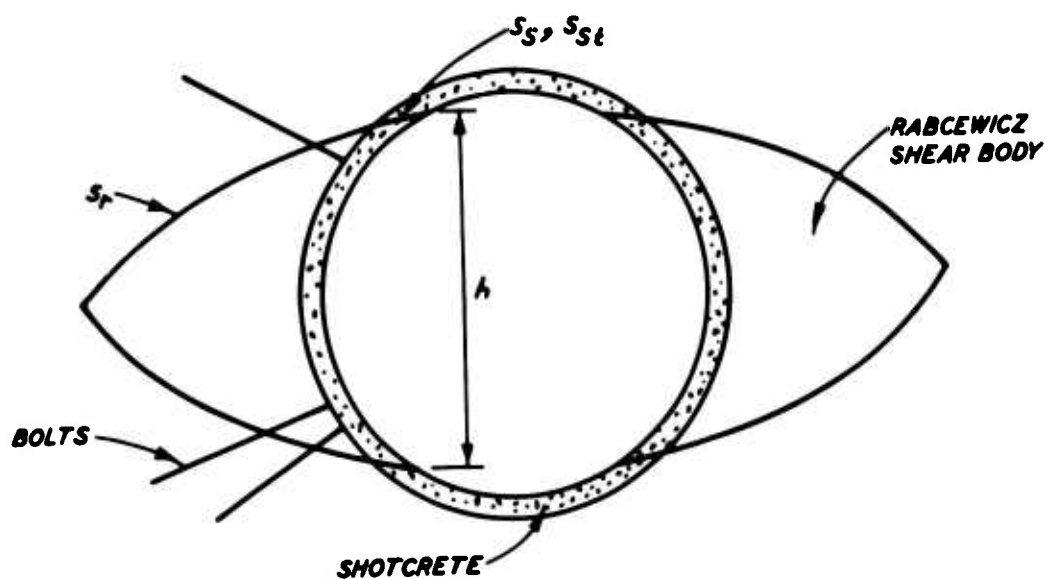


Figure 13. Determination of p_i minimum



LEGEND

- S_r = SHEAR FORCE ALONG BOUNDARY OF "SHEAR BODY"
- S_s = FORCE FOR SHEARING THROUGH SHOTCRETE
- S_{st} = FORCE FOR SHEARING THROUGH STEEL SETS AND THROUGH REINFORCING STEEL
- h = HEIGHT OF SHEAR BODY

Figure 14. Force equilibrium analysis for Rabcewicz shear body

Initially, readings are taken every other day, decreasing in frequency to once per month. The time-displacement curves are used to decide on additions or reductions of this temporary support and on the point in time when the final support (liner) can be constructed. The decision on additional temporary supports is also extensively based on observations by the tunnel crew. It was stated that tunnel crews familiar with the NATM are "enthusiastically" contributing to its success by careful installation of the instruments and observing and reporting the support behavior. It is the goal of the NATM to use temporary support systems that are as close as possible to a factor of safety of 1, i.e. just preventing loosening. The final liner is constructed after the displacements have leveled off. The final design and construction are thus intrinsically tied together. The contractor plays an important part in the decisions; since he is paid for the additional work involved, he contributes willingly. A summary of a case history may provide further illustration of the NATM. In the Tauern highway tunnel (horseshoe, diameter approximately 10 m) in Austria, a heavily fractured phyllitic rock was encountered for a length of 300 m. The predesigned support system consisted of shotcrete thickness $d = 15$ cm, 4-m bolts spaced 0.75 m, and light (25 kg/m) steel sets. In most of this zone, maximum total displacements of less than 25 cm took place, which could be tolerated with some repairs. In a short section of 40 m in this zone, the displacements of 80 cm led to extensive destruction of the shotcrete and distortion of the steel sets. Based on these observations, the following support system shown in Figure 15 was designed and used. Reinforcing steel wire mesh was installed, and shotcrete sprayed immediately after excavation. Axis-parallel slots of 15-cm width were cut at distances of 2 to 3 m along the tunnel surface. Steel sets were installed and longer bolts were used (Reference 15). The rock deformed, closing the slots, deforming the reinforcing steel and distorting the steel sets (could be tolerated). The deformations leveled off after closure of the slots. This example is probably the high point of engineered construction and a prime example of the observational method. The NATM has been successfully used in soft ground also. Examples include the Frankfurt subway (soft-shale-clay) and the entrance sections of the Tauern tunnel. In soft ground, it will be frequently necessary to reduce the cross section of the initially excavated tunnel

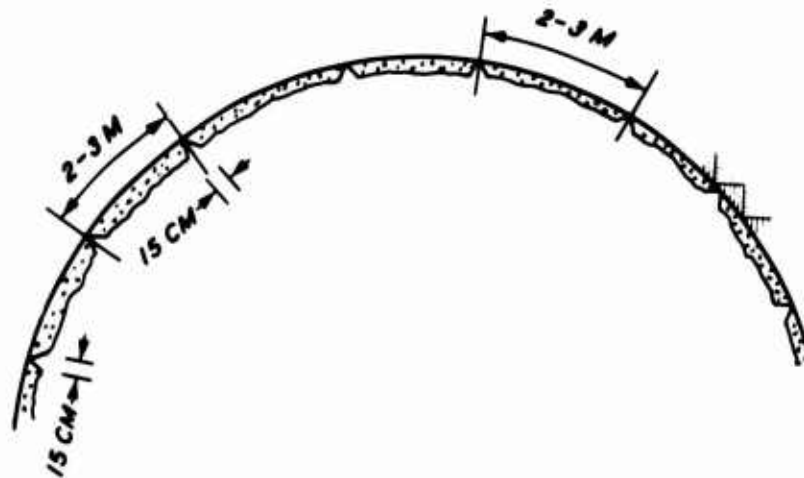


Figure 15. Cross section of the
Tauern tunnel

section by heading and benching or multiple drift procedures in order to provide sufficient standup time for support installation.

- e. Conclusions. One purely empirical and three analytical approaches with empirical parts of varying importance have been described. All these approaches take the contribution of the ground in the ground-support system into account. This contribution is particularly appropriate in the case of shotcrete support. Therefore, none of the traditional "rock-load" approaches have been shown here. The flexibility of tunnel supports is a logical outgrowth of this ground-structure interaction concept. However, not only flexibility but also prevention of excessive displacements is an important characteristic of modern tunnel supports. Continuous support as provided by shotcrete and closure of the support at the invert can therefore be of great importance. The empirical input is significant in all cases, and this may be one reason why the approaches can at the present time only be successfully used by experienced designers. The necessity for empirical input is caused by the complex behavior of the ground-support system which, particularly in the case of shotcrete, consists of many interacting mechanisms. A description of the behavior by traditional geotechnical parameters is insufficient, and extensive observations in situ are necessary. These observations may be either used as parameters in the analysis or directly included in the design-construction procedure or both. The full success of integrated ground-support systems and particularly shotcrete depends to a large extent on the contractor's cooperation and contributions. It is interesting to notice that all the described analytical

approaches make use of the characteristic curves as a means of describing ground-support behavior. The temporary character of modern shotcrete supports justifies both the important role of empirical design approaches and the design "close to a factor of safety of 1." Since failures usually do not have catastrophic consequences, they can be overcome by repair of the failed section and modification of the design of subsequent sections. Sophisticated analytical methods are used to some extent, for example, in the numerical approximations for the elastoplastic stress computations or FE analyses for the computation of characteristic curves without time and volume effects. In other words, the advanced analytical methods are only used for certain portions of the tunnel-support analysis, not only due to the difficulty in determining appropriate input parameters but also due to the fact that the ground-support behavior is not well understood and thus not well represented in the conceptual models. It should be kept in mind that the discussions on support design and particularly on shotcrete supports apply to rock and soft ground, provided the latter has a comparable standup time.

48. Detailed support design. As has become clear in the previous discussions, this detailed support design is a part of the design of the ground-support system, e.g., the characteristic curves of the support or the resisting forces in the NATM are based on detailed support dimensions and material properties. In the traditional support design approaches, e.g. the "rock load approach," detailed support design is not integrated but follows the load determination. In the NATM shotcrete, reinforcing steel and steel sets are designed against shear failure, and rockbolts are designed against yielding in tension.

- a. The flexible temporary supports in the "characteristic curve" and "combined" approaches are usually designed based on the hoop stress formula. Steel sets are considered "smeared out" over the tunnel surface. Additional checks are made for the support of individual rock blocks (Figure 16) in an approach analogous to the force equilibrium of Rabcewicz shear body, i.e., shearing along rock interfaces and through supports and tension forces in bolts. (The design against individual blocks could also be incorporated in the NATM, but contradicts to some extent the basic concepts of this method.)
- b. Rigid supports (temporary and final) are analyzed as rings or arches resulting in thrust forces, moments, and displacements. Steel sets are considered as smeared out

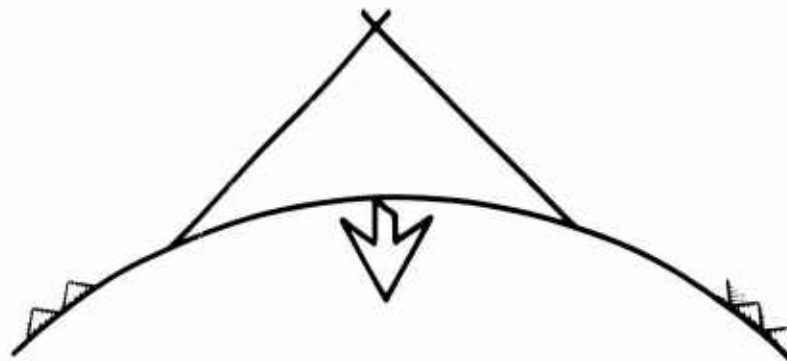


Figure 16. Failure of individual block

reinforcements. Using thrust-moment interaction diagrams (Figure 17), reinforced concrete supports are usually designed based on the ultimate strength approach. Since the support performance has to be used in the characteristic curves, it is necessary to construct such curves for various support characteristics. This can be done by closed form solutions, the stress method, or FE methods. The stress method actually allows to some extent an integrated ground-structure analysis using ground reaction coefficients.

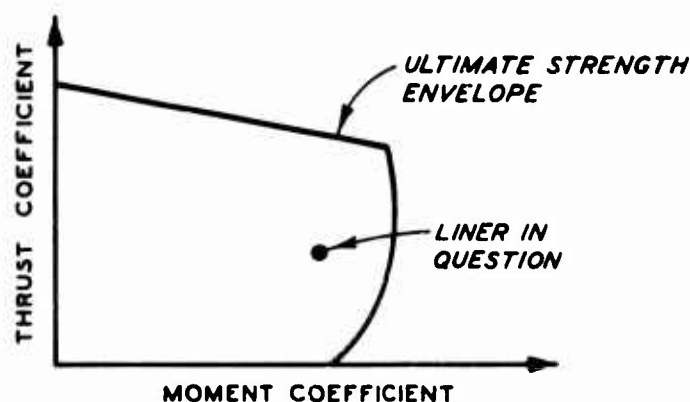


Figure 17. Thrust-moment interaction diagram

Nonstructural shotcrete in tunnels

49. Shotcrete for sealing purposes. Shotcrete can prevent water and gas flow due to its low permeability. The use of shotcrete as a water stop is frequently made during construction and is not specifically designed. Shotcrete can be applied directly on surfaces through which distributed "area" flow enters the tunnel. (Relief holes are drilled later through shotcrete into the ground.) By adding more accelerator and increasing the cement content, rapid setting and strength increase

can be achieved; this modification of the shotcrete mix is frequently decided on the spot. Where such flow conditions are frequent, a 1- to 2-cm layer of Gunitite is applied first before spraying shotcrete. If concentrated flow occurs, plastic hoses are inserted at the location before shotcreting. This method has been expanded to a drainage-water stop system, which is to some extent designed as follows: plastic channels of half-circular cross sections are placed at regular intervals with the open face against the rock and their lower end connected to the tunnel drainage system. Concentrated flows are diverted with plastic tubes into these channels. The channel-tube system is sprayed over with shotcrete. The number and size of the channels depend on the estimated inflow. Linder (Reference 16) describes a similar system using prefabricated concrete elements. The reduction of slaking and swelling with a thin 5- to 10-cm shotcrete layer is usually decided on the spot.

50. Shotcrete as support of water stop. Water stops in the form of sprayed or rolled-on liquids can normally be applied only on a dry and reasonably even surface. Water stops in the form of sheets have to be welded against point or strip type support, which in turn have to be fixed to the tunnel surface. Both kinds of water stop should be protected from puncturing by sharp edges. Shotcrete is ideally suited for all these purposes. Its thickness depends on the unevenness of the natural tunnel surface. From a purely structural point of view--holding the water stop in place--a layer of 2 cm (Gunitite) is sufficient. Additional thickness is, however, necessary to even out the natural surface.

51. Shotcrete for the protection of steel support. The Bernold method consisting of perforated corrugated steel sheets backfilled with concrete (Reference 17) requires a shotcrete protection of the steel sheets. Steel sets that would be exposed for a long period of time can also be protected by shotcrete in a 1- to 2-in.-thick layer. Although mainly intended for corrosion prevention, shotcrete contributes to the structural support performance in all these applications and may thus be included in the design.

Other shotcrete applications

52. Applications like dish- or tub-shaped structures in the ground

or slope protection are designed as for normal concrete. As mentioned before, these applications are taking advantage of the formwork elimination by shotcrete and the mechanization of concrete placement, rather than make use of unique shotcrete characteristics in the ground-structure interaction mechanism. However, there is actually no reason why those characteristics should not be taken into account in the design particularly of slope protection. The author of this report does not know if such a design approach has been used. In the area of "shotcrete structures" or "sculptural applications," the design particularly from an architectural point of view has definitely made use of the particular shotcrete characteristics. (Several modern churches have been built mainly by shotcreting.)

53. This concludes the discussion on shotcrete design. It has been shown that shotcrete has unique characteristics: the possibility of constructing thin and thus flexible supports, its continuous character in contrast to other flexible supports, the interaction mechanisms between shotcrete and ground, the possibility to combine it with other supports, and its low permeability. These unique characteristics have also been incorporated in the modern design approaches previously described. Since some of these characteristics are strongly influenced by the shotcrete technology, it seems warranted to discuss the most important technological aspects.

Shotcrete Technology

54. Three major problem areas are mixture design, shotcrete application, and control. This discussion is based on the use of shotcrete in tunnels only.

Mixture design

55. Aggregate. Maximum grain-size is 30 mm, sometimes 35 mm, but frequently only 15 or 20 mm. Most guns have their maximum performance for a maximum grain size of 20 mm. Maximum grain-size Gunitite is 6 or 8 mm. No broken material should be used since it has the tendency to break down further during spraying. Equidimensional particles are

preferred since they can be placed in a texture of higher density compared to other particle shapes.

56. Cement control. Shotcrete is 300- to 400-kg/m³ dry mixture; Gunitite is 400- to 500-kg/m³ dry mixture. Usually, cement is type III. (A new development is the specification of cement content of the shotcrete in place and to leave it to the contractor to select the proper dry mixture. In one application, 350 kg/m³ in place was specified, which was achieved with a dry-mixture cement content of 270 kg/m³.)

57. Water-cement (W/C) factor. Values vary between 0.55 and 0.30, smaller values for greater maximum grain size. It should be noted that the rebound decreases with increasing water content. Maximum strengths are reached for W/C at approximately 0.35, decreasing to 80 percent of the maximum for W/C ~ 0.55.

58. Accelerators. Powder type--between 2 and 7 percent of cement weight. They frequently do not get evenly distributed in the mixture and are therefore decreasingly used. They should not be used in wet-mixture shotcrete.* Liquid type (Citodur, Sarabond)--approximately 6 percent of cement weight.

Shotcrete application

59. Surface treatment. The ground surface has to be cleaned. Sandblasting may be used, but due to the danger of silicosis, it is frequently replaced by high-pressure water jets. The smooth surfaces of machine-driven tunnels reduce the adherence of shotcrete, particularly in wet conditions and low-strength ground (shale). Under such circumstances, wire mesh (in Europe usually 10 by 10 cm of 8-mm reinforcing steel) should be first fixed to the rock surface even if it is not needed for structural reasons. The wire mesh is fixed by short bolts. An insufficient number of fix points leads to vibrations of the mesh and loosening of already sprayed shotcrete. The number of bolts has to be determined on site, since the vibrations depend on the application parameters (distance, velocity, and direction).

60. Shotcrete (Gunitite) layers. Usually, an initial layer richer in cement is applied. For example,

* The new equipment by Challenge-Cook Brothers has, to the author's knowledge, not yet been used in Europe.

- a. Guniting initial layer, approximately 1 cm thick, cement content 500 kg/m³, dry.
- b. Shotcrete initial layer approximately 2- to 3-cm Guniting; applied according to the criteria mentioned above.

The following layers can be applied without treatment of the surfaces of previous layers. The thickness of shotcrete that can be applied at one time depends on the use of accelerators. Shotcrete without accelerators cannot be applied overhead in thicknesses greater than 3 cm (Reference 16). With accelerators there are practically no limitations. The layer thickness is then governed by length of tunnel section, available time, and performance of the equipment, keeping in mind that it is more important to cover the tunnel surface with a thin layer as rapidly and completely as possible than to spray thick but limited shotcrete patches. (Wire mesh (10 by 10 cm, 8-mm rebars) mats are of a width corresponding to the section to be shotcreted (round length) but for reasons of handling are rarely wider than 1 m. These mats are usually fixed directly to the rock, but sometimes to the initial Guniting or shotcrete layers.)

61. Spraying techniques:

- a. A gun is held at a distance of approximately 1 m from the surface, perpendicular to the surface. Spraying has to proceed from lower to higher parts of cross section to prevent the embedding of rebound in the lower parts.
- b. Rebound depends on temperature (optimum 15 to 25°C), cement content (less rebound if high), W/C factor (less rebound if high), maximum grain size (most for approximately 15 mm), jet velocity, and direction of jet relative to surface. The rebound contains more coarse particles and less cement than the initial mixture, thus implying that the shotcrete in place has a smaller maximum grain size and higher cement content than the specified mixture.
- c. The equipment is usually the dry-mixture type. Wet-mixture equipment is, however, used as high-rate concrete placement equipment in connection with the Bernold method.
- d. Remotely controlled equipment includes truck- or rail-mounted equipment with booms or rotating arms supporting one or several nozzles (Reference 18), and particularly is used in large-diameter tunnels where spraying would have to proceed from special scaffolding or jumbo decks. The author is personally familiar with one case in which the remote control limits the close observation

of the spraying operation and thus the fine regulation that a hand operator normally provides. Consequently, rebound (up to 40 percent +) and the embedding of rebound were significantly increased. This may, however, have been an exceptional case.

62. Other problems:

- a. Distance from tunnel face. The requirement that shotcrete be applied close to the tunnel face in low standup time rocks may not be fulfilled in tunnels driven with the tunnel boring machine (TBM). Only large-diam (>6 m) machines permit the installation of shotcreting platforms or a remotely controlled shotcreting equipment directly behind the rotating head (which still means that there is an "unsupported" section of about 1 m +).
- b. Blasting effects. Shotcrete reaches sufficient strength to permit blasting of a new round, approximately 10 hr after spraying up to the face. Damages due to earlier blasting can be repaired by spraying. With smooth blasting or line drilling, blasting can proceed 2 to 3 hr after spraying.
- c. Rebound removal. Rebound removal causes problems in addition to the increased muck removal and transportation capacities if shotcreting does not occur in the zone where mucking takes place. Additional loaders have to be provided and constant cleaning of the roadway or track leading through this section are necessary. In TBM driven tunnels, the shotcreting will always be at a different location from the muck loading. Also, TBM and their follow-up equipment (conveyor belt gantries) roll or slide on the invert or on special tracks on the invert and are thus affected by the debris on the invert; the space between TBM and invert is limited, thus making cleaning operations difficult.
- d. Invert shotcreting. The requirement that the support be closed to a full ring as soon as possible causes difficulties because of the installations that have to be placed and the equipment that are moving on the invert. Even the short setting and strength-increase time may be too long from a construction operation point of view. Shotcreting of the invert is definitely faster than the frequently specified cast-in-place concrete, but may have to be substituted by prefabricated elements if it impedes the construction operation.

Control

63. This problem area involves only the control of the shotcrete and not the control of the ground-support system (previously discussed

in paragraph 47). The two main aspects that have to be controlled are as follows:

- a. The shotcrete thickness is controlled by steel or wooden pegs; often bolt heads may serve the same purpose. New surveying techniques making use of photogrammetry can detect differences of ≥ 1 cm.
- b. The shotcrete quality in place cannot be controlled in a satisfactory manner at the present time. Core specimens are limited in number; index tests (indenters, rebound hammers) have to be conducted in the fresh shotcrete (indenter) and leave uncertainties about the final quality, or they involve both ground and shotcrete (rebound hammer). At the present time, only the quality of the raw material can be stringently controlled, which is unsatisfactory in view of the importance of the application parameters.

64. This phase of the report on shotcrete technology shows that the technological aspect of shotcrete design involves considerable problems that warrant attention. It also emphasizes the importance of the construction crew and particularly of the nozzle operator, again confirming the dependence on contractor cooperation.

65. The discussion of European shotcrete design has to include a few words on developments that may affect the use of shotcrete in the future.

Developments Affecting Shotcrete Design and Practice

Developments in analysis and design

66. ETH--Zurich, Switzerland--has developed an elastoplastic finite element program for tunnel design. It has been practically used for about 4 yr. Problems with regard to input parameters and particularly the modeling of the actual behavior (the "unloading mode" due to excavation is only approximated) remain.

67. ETH--Lausanne, Switzerland--is modifying a three-dimensional elastoplastic FE program that has been developed for reactor design. A few test runs have been compared with observations in tunnels and show

satisfactory results. The program needs at least 2 to 3 yr of further development.

68. Extensive measurement programs are presently under way also in tunnels not constructed by the NATM. The main goal of these programs is the widening and refinement of the data base of the design methods described in paragraphs 47 and 48. The measurements involve: changes of the stress state and displacements as the tunnel advances to and passes the measurement sections; extent of the plastic zone and particularly of the plastic volume increase.

Developments in
material technology

69. (The author has written a report on potential advances in concrete technology applicable to tunnel supports for the Swiss Federal Railroads from which some of the following statements are taken.)

70. Regulated set cement. Seemingly ideally suited for shotcrete in providing early strength and practically no reduction in final strength (in contrast to accelerators), it is also cost competitive. There are, however, some problems that need further investigation. Shrinkage cracks due to rapid setting and associated heat development have to be avoided since they defeat some of the main purposes of shotcrete, such as continuity of the support. Also, the strength increase may actually be too rapid since some "creep" behavior is actually part of the shotcrete-ground interaction mechanism. This requirement may be satisfied by modifying the cement characteristics.

71. A thorough investigation is also necessary of how all the conditions in a tunnel (temperature, moisture, ground characteristics) affect the regulated set cement. Furthermore, if an in situ modification of the set characteristics could be provided, this could be incorporated in the in situ design modifications of the NATM. However, the cement industry seems reluctant to push regulated set cement.

72. Fiber reinforced concrete. Steel or glass fiber shotcrete would allow a faster placement of reinforced shotcrete than the present procedure. The author does not know of any commercially available equipment or any practical application. The cement industry has, however,

extensively experimented with this method. Problems are the distribution and orientation of the fibers in the shotcrete. (Will the fibers be paral'led to the tunnel surface? Will they increase the rebound due to their flexible characteristics?) Normally, reinforced shotcrete is less expensive than fiber reinforced shotcrete.

73. Ferrocement (only in connection with Guniting). This is one of the most promising methods to produce a high-tensile-strength concrete. Such a material can be used in thin layers (approximately 2 in.), can provide sufficient resistance, and is at the same time highly flexible, thus conforming well to the support requirements previously outlined. The disadvantages are the large labor component in the total cost and the mesh size that requires Guniting and makes the adaptation to an uneven tunnel surface difficult.

74. Polymer-impregnated concrete, polymer-cement concrete, and polymer concrete. Polymer impregnation is probably not possible due to the usually low permeability of shotcrete and the difficulties of impregnation and catalyzation procedures in the tunnel environment. Polymer concrete will basically have higher compressive and particularly higher tensile strength than cement concrete. These desirable characteristics are presently outweighed by sensitivity to moisture and cost.

75. Prefabricated elements. The problems of shotcrete application in TBM driven tunnels (application directly behind the face is impossible; rebound disrupts the TBM operation and requires additional removal equipment) lead to an increased use of prefabricated elements, particularly large precast concrete elements (five to nine per circumference) that have been developed and are extensively used in Switzerland. Developments like ferrocement and polymer-impregnated concrete may make the flexibility and installation time of supports composed of such elements equal to or better than those of shotcrete supports.

76. Shotcreting equipment. The remotely controlled guns have already been mentioned. Their use seems to be promising once their application quality becomes similar to hand application.

77. A logical extension of the subjects just discussed of the recent or near future developments is a few comments and recommendations for research and development.

Recommended Research and Development in Both
Design-Analysis and Technology

Recommended research and development in analysis and design

78. The mechanisms governing shotcrete behavior (see paragraphs 38-43) seem to be recognized, but their relative contribution and also the question if other mechanisms are involved have to be further investigated. This may only be possible in the context of the treatment of the following more basic problems.

79. The ground-support behavior in tunnels is very complex, and the analytical methods include many approximations. A better understanding of this behavior obtained from measurement programs and reviews of case studies is necessary.

80. With improved understanding of the ground-structure behavior, it will then be possible to improve the analytical methods (e.g. FE methods) and particularly to determine appropriate tests for the input data of the analyses.

81. It may not be possible or desirable to reduce the substantial empirical parts of the analysis-design procedure. However, it is necessary to clearly establish the need for empirical input based on the improved understanding of the ground-structure behavior. Also, once the need for empirical input has been established, it is important to formalize it; that is, to tell the designers where in the design process it has to be used and on what it has to be based (in situ measurements, data from other tunnels).

82. The description of the design methods, particularly the comments on the NATM, indicate that improvements in the usual United States tunnel design and construction procedures* could be achieved by applying

* Although progressive design and construction procedures are practiced in Europe, there are still many tunnels designed and built in the traditional way.

these methods. This will be difficult, however, due to substantial deviation from the present design procedures, specifications, and contractor-engineer-owner relations. A test tunnel or test section in a tunnel may be useful to promote these changes.

Recommended research
and development in technology

83. A major problem is the quality assessment of shotcrete in place and the relation between shotcrete characteristics in place, ground characteristics, raw material, and application procedure.

84. Instrumentation and measurement techniques for shotcrete quality assessment and for the monitoring of ground-support behavior need to be developed.

85. Material and application technology could be pushed further in the direction of "ideal support behavior:" immediate support application after excavation; controllable support displacements that optimize the ground contribution. Research and development should thus consider high-strength, low-cost materials with adjustable flexibility. This may be done by pursuing some of the promising developments outlined in paragraphs 66-76.

86. The mechanized application equipment should be developed such that continuous monitoring and adaptation of in situ shotcrete quality and layer thickness are possible.

87. In some cases, other types of supports may be better suited than shotcrete or its combinations with other supports. Particularly, the development of prefabricated liners should be pursued.

Summary and Conclusions

88. The review of European shotcrete design and practice showed that shotcrete is used: (a) in tunnels--as structural support, as a seal to prevent alterations of the ground and to reduce water inflow, and as support for water stops; and (b) in surface applications--as structural member where use of formwork would be difficult or more expensive, and as slope protection or where irregular surfaces are required.

89. These shotcrete applications involve either the structural performance of the shotcrete itself, the structural performance of shotcrete and ground acting together, or its characteristically low permeability. The uniqueness of shotcrete comes to bear in the ground-shotcrete interaction, which is of particular importance in the design of tunnel supports.

90. The modern design approaches that have been discussed in this report all take the ground-structure interaction prominently into account. Closed form and numerical analyses, both making use of the computer, play an important role in the described design methods. Their use is, however, limited because of the lacking data base for input parameters and the inadequacy of the underlying conceptual models. The most important characteristic of these design methods and particularly of the New Austrian Tunnel Method is the engineered construction, i.e., the design methods incorporating empirical input and particularly observations made during construction. This allows the designer to stay close to a factor of safety of 1 for the temporary supports. Close cooperation of the contractor is necessary.

91. The main advantages of shotcrete as a tunnel support are, on the one hand, its flexible character (that permits ground displacements and thus mobilization of the ground resistance, which thereby contributes to the support action) and, on the other hand, its continuity and the possibility of rapid application (which minimizes the possibility of "loosening" and the associated reductions in ground resistance). The prevention of loosening may require closing the shotcrete support to a full ring.

92. Comments on shotcrete technology, that is, mixture design, application techniques, and control, show that a great number of factors govern the quality of the shotcrete in place.

93. Research and development is necessary in the areas of ground-support behavior, detailed mechanisms involved in the shotcrete behavior, improvement of material characteristics and quality control, and possible replacement by other support types.

PART III: USE OF SHOTCRETE FOR TUNNEL LINING

by

U. S. Bureau of Reclamation
Engineering and Research Center*

Introduction

94. There is considerable interest today in the use of shotcrete for tunnel linings. Indicative of this was the 1973 Shotcrete Conference (American Concrete Institute Publication SP-45; Reference 19) held in South Berwick, Maine, and the followup conference scheduled for the fall of 1975. Committee 506, Shotcreting, of the American Concrete Institute (Reference 20) is presently considering preparation of a state-of-the-art report on shotcrete for underground support. The Civil Engineering Department at the University of Illinois, under contract to the Federal Railroad Administration, has prepared a shotcrete handbook for designers and field personnel (Reference 21).

95. The Corps of Engineers has used shotcrete for tunnel linings and other underground support applications and like others, including the Bureau of Reclamation (USBR), has experienced varying kinds and degrees of difficulties. To obtain a better insight into the solution of such difficulties and to help determine where research dollars should be expended, the U. S. Army Engineer Waterways Experiment Station (WES) at Vicksburg, Mississippi, has assembled this state-of-the-art report on shotcrete design and practices. In June 1974, the USBR entered into an agreement with the WES to prepare this section of the report on the USBR's design and construction concepts and experiences in using shotcrete for tunnel linings.

96. Although, for a number of years, some form of shotcrete has been used in USBR tunnel construction, it has only been in recent years that shotcrete has been used for primary tunnel support. Past usage

* This report was prepared by engineers and technicians at the Bureau of Reclamation, Engineering and Research (E&R) Center in Denver, Colorado.

generally has been in conjunction with steel rib supports for protecting rock surfaces from deterioration and sloughing. More recently, the USBR has used shotcrete alone for temporary support, with a concrete lining installed at a later time. Experiences gained from these uses have given designers confidence in the use of the shotcrete support system, and it is now being used alone as primary support and final lining where justified. Cunningham Tunnel, an 8-ft-(2.4-m-)diam, gravity-flow, water conveyance tunnel presently (1974) under construction on the Fryingpan-Arkansas Project in Colorado, will have parts of its 2.8-mile (4.5-km) length lined with shotcrete for total support and final lining. A more detailed report of shotcrete work in this and other tunnels is described later.

97. The USBR's philosophy regarding the use of shotcrete in underground tunnel work is much the same as it is for other design and construction activities. Since much of the cost for design and construction of tunnels and other project features must be repaid by those who benefit from the structure, designs must be as economically attractive as possible while not sacrificing aesthetic values and safety. This practice is best demonstrated by the use of alternate bidding schedules where, for instance, the use of shotcrete for final tunnel lining is permitted at the contractor's option.

Design Considerations and Experiences

Shotcrete uses specified by designers

98. Shotcrete is defined (American Concrete Institute) as "mortar or concrete conveyed through a hose and pneumatically projected at high velocity onto a surface." Under this broad definition, shotcrete composed of various combinations of cement, aggregate, and accelerator has been specified for the following purposes on USBR underground shafts, tunnels, and chambers:

- a. Primary support and final lining.
- b. Protective coating on excavated surfaces of tunnels and underground chambers over materials that deteriorate when exposed to air. This coating provides temporary protection until the final concrete lining is placed.

- c. Protective coating over structural steel rib supports, rockbolt plates, heads and nuts, associated metal shapes required for support of underground chambers, and wire fabric used to prevent rockfalls.
- d. An alternate for timber, steel, or concrete lagging between structural steel rib supports.

99. Table 1 lists recent USBR construction where shotcrete has been used and describes the purpose of the shotcrete applications. No shotcrete has been used in USBR "soft ground" tunnels where liner plate has been required for support. The chief advantages of shotcrete as a supporting or protecting medium in construction work are (1) the speed at which it may be applied in amounts and locations as needed, and (2) the economy resulting from the continuous construction process. USBR design drawings and narrative specifications are tailored to assure that adequate thicknesses of shotcrete are placed in critical areas at critical times.

Design procedures

100. Designs for underground facilities usually begin with a study of geologic data. These data customarily include a map of the surface geology above the facility, the logs from drill holes, and the rock cores drilled on or near the construction site. At the conclusion of the geologic study, the designer usually decides on whether the excavated opening must be completely lined or whether parts may be left unlined. Except for tunnels constructed in the early 1900's, the great majority of USBR water conveyance tunnels built prior to 1970 were completely lined with concrete (Reference 22). After 1970, in an effort to reduce tunneling costs, tunnels were designed to include linings only where they were absolutely required and where practicable. This criterion applied particularly to long, gravity-flow, water collection tunnels (Nast, Hunter, and Cunningham Tunnels of the Fryingpan-Arkansas Project in Colorado). These tunnels, excavated through granite and gneiss formations, were not lined where competent, erosion-resistant rock existed, and shotcrete was used for primary support and final lining wherever possible. At this time (1974), the first of these tunnels has been completed but not yet subjected to design flows. The

Table 1
Summary of USBR Tunnels with Shotcrete Linings

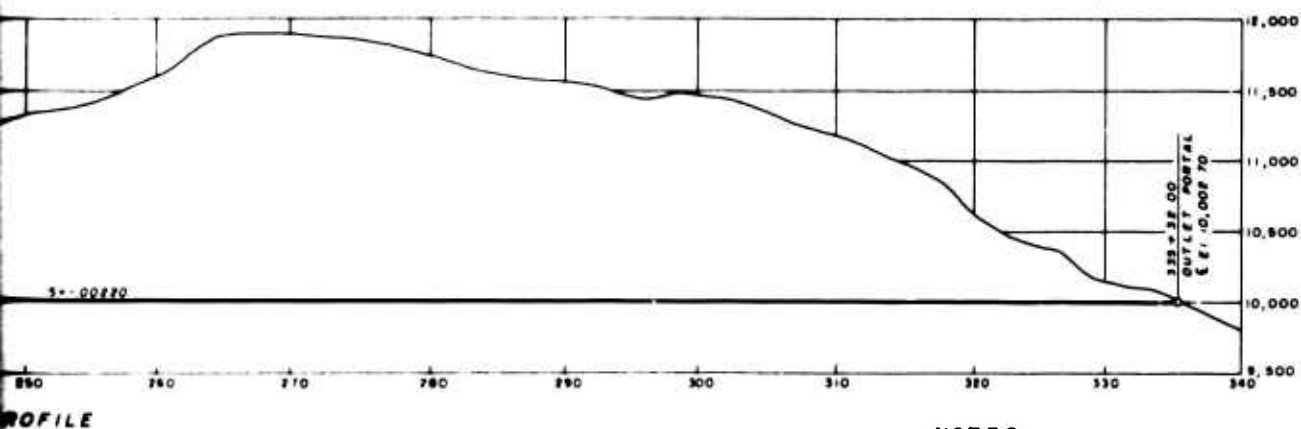
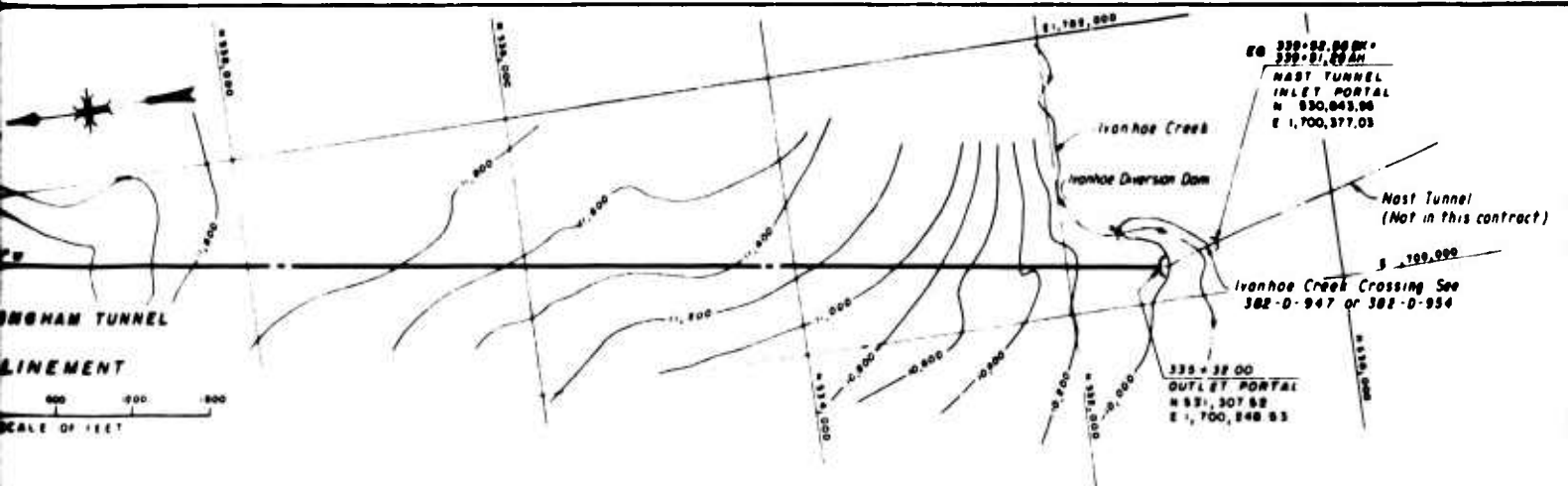
Specifications No. DC-	Feature	Project	State	Date Work Started	Shotcrete Use
7060	Auburn Dam Exploratory Tunnels	Central Valley	CA	1974	Replacement for existing steel rib supports. Final tunnel lining
7024	Cunningham Tunnel	Fryingpan-Arkansas	CO	1973	Primary support and final tunnel lining
6983	Tunnel No. 4	Navajo Indian Irrigation	NM	1973	Protective coating over shale. Beneath final concrete lining
6952	Scoggins Dam-Outlet Works Tunnel	Tualatin	OR	1972	Protective coating and primary support. Beneath final concrete lining
6849	Tunnels No. 3 and 3A	Navajo Indian Irrigation	NM	1970	Protective coating over shale. Beneath final concrete lining
6839	Hunter Tunnel	Fryingpan-Arkansas	CO	1970	Primary support and final tunnel lining
6829	Nast Tunnel	Fryingpan-Arkansas	CO	1970	Protective coating over tunnel support metalwork. Final lining over erodible rock seams
6651	Intake Tunnel and Pumping Chamber	Southern Nevada	NV	1968	Protective coating and fallout prevention. Final lining used with wire fabric and rockbolts
5575	Water Hollow Tunnel	Central Utah	UT	1968	Protective coating over excavated surface. Beneath final concrete lining
6429	Gunnison Tunnel Rehabilitation	Uncompagre	CO	1966	Protective coating over excavated surface. Beneath final concrete lining
6380	Oso Tunnel	San Juan-Chama	NM	1966	Protective coating over excavated surface. Beneath final concrete lining
6277	C. H. Boustead Tunnel	Fryingpan-Arkansas	CO	1966	Temporary support in shear zone. Beneath final concrete lining
6261	Blanco Tunnel	San Juan-Chama	NM	1965	Protective coating over excavated surface. Beneath final concrete lining
6070	Azotea Tunnel	San Juan-Chama	NM	1965	Protective coating over excavated surface. Beneath final concrete lining

other two are under construction. Therefore, performance data pertaining to recently constructed, partially lined tunnels are lacking.

101. In estimating the type and extent of support required in underground openings, consideration is given as to how the facilities will be excavated. In earlier years, underground facilities were conventionally excavated in rock by the drill and blast (DB) method. However, in the past 10 yr, 12 long USBR water conveyance tunnels ranging from 9 to 21 ft (2.7 to 6.4 m) in diameter have been excavated by machine boring (Reference 23). Alternate bidding schedules are usually provided in USBR specifications for long water conveyance tunnels that allow a contractor to select the least costly method of excavation for the particular job. Figures 18 and 19, taken from Cunningham Tunnel Specifications No. DC-7024, show the required tunnel sections deemed acceptable under either DB or machine-bore (MB) method of excavation for the particular geologic setting traversed by this tunnel.

102. Although considerable progress has been made in equipment development in the last few years, most TBM's are not presently equipped to apply shotcrete for tunnel support, especially in small (15-ft (4.6-m) diameter and less) tunnels. The principal difficulty is that adequate space is not available around the machine and its trailing equipment to permit shotcrete applications immediately behind the cutterhead. Another problem is the inability of thin shotcrete tunnel lining to withstand thrust from the boring machine support pads.

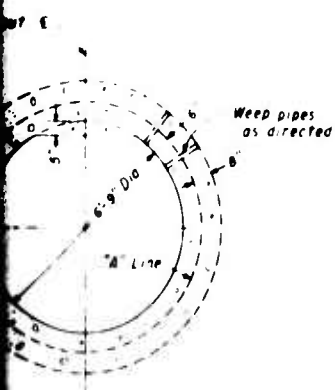
103. Specifications for ten of the twelve MB tunnels (Reference 23) allowed the contractor the option of using shotcrete for protective coating in nine tunnels and for primary support in one. Contractors elected to use protective coating shotcrete in six of the tunnels, but no primary support shotcrete has been used in any MB tunnel to date (1974). In several instances, however, USBR specifications have included requirements for shotcrete in MB tunnels where application could be carried out near the end of the conveyor that trails the TBM. In those instances, shotcrete application was used principally to control deterioration of the excavated surfaces resulting from stress relief or moisture changes.



NOTES

Coordinates shown refer to the Colorado State Plane Coordinate System, Central Zone, as established by the Coast and Geodetic Survey. Stations shown have been adjusted to give true horizontal distances at a mean elevation of 10,000 feet. Stations and elevations shown on alignment and profile refer to center line of tunnel. For details of typical tunnel supports, see 302-D-942. Concrete design of formed tunnel lining based on a compressive strength of 3,000 pounds per square inch at 28 days.

Minimum length of concrete lined section will be 32 feet. Where concrete lining is placed in an unsupported or rock bolt supported section the finished diameter of the tunnel shall be the same as for the rib supported section.



RIB SUPPORTED
CONCRETE LINED SECTION

HYDRAULIC PROPERTIES

SECTION	A	V	Q	F	n	s	d
Unlined	42.02	6.43	270	2.50	0.020	0.022	5.94
Concrete lined	31.19	8.66	270	2.05	0.013	0.022	5.49

These properties based on a normal depth in the section

EXCAVATION AND CONCRETE QUANTITIES

Quantities in cu yds per linear foot

SECTION	EXCAVATION	CONCRETE
Unlined	2.400	—
Concrete lined	2.400	1.109

alignment, profile, and sections for machine-bored tunnel

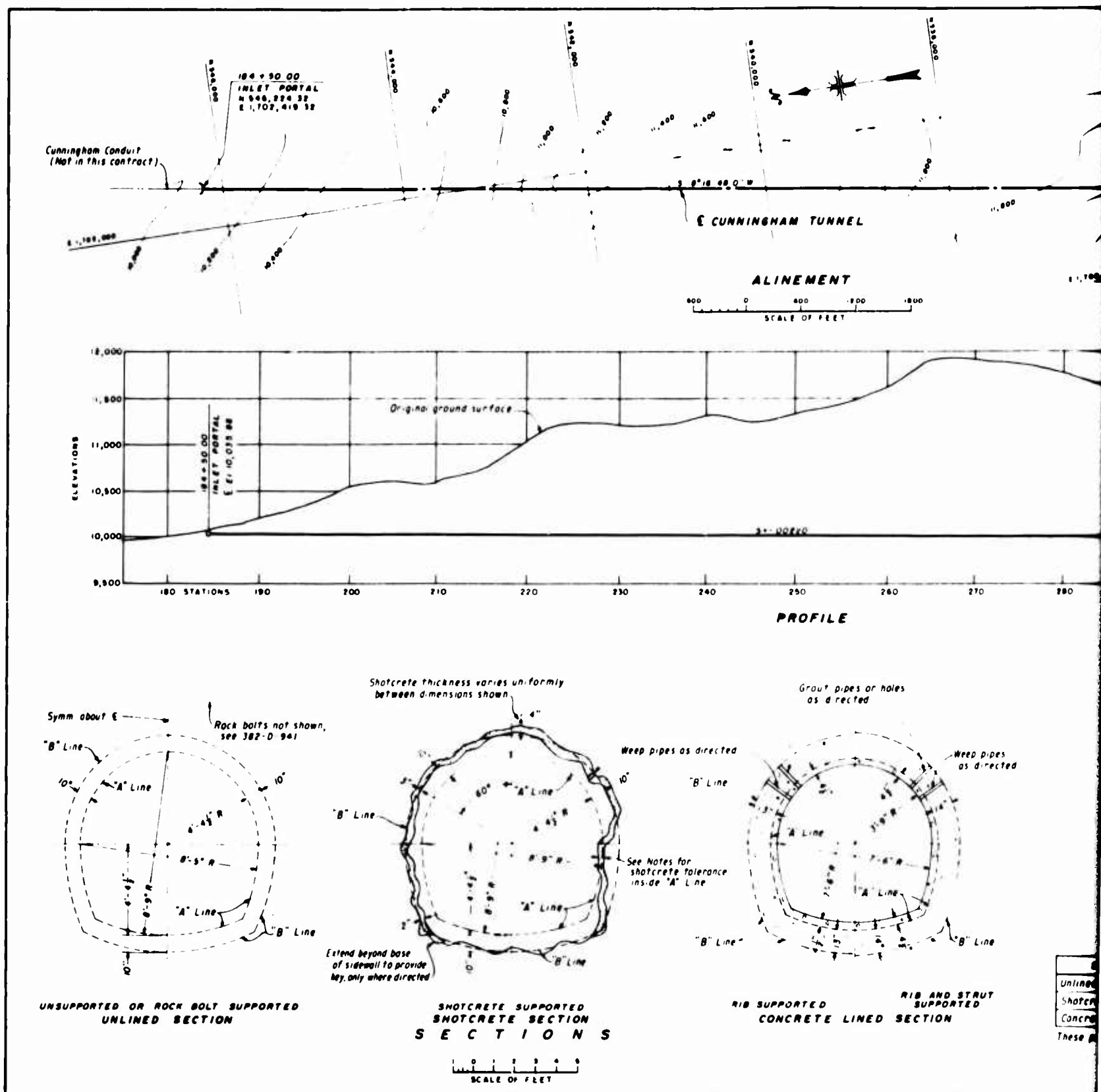


Figure 19. Cunningham Tunnel--alignment, profile, and sections

104. In preparing specifications for tunneling work, the designer must constantly be aware of the space limitations that dominate underground operations. Space limitations are especially critical in small-diameter tunnels. For such tunnels, it is particularly important that specifications requirements for supports be clearly defined. Doing so will permit a contractor to bid on a tunnel with some assurance of the exact type of construction activity that will develop in the tunnel.

105. The service conditions under which the tunnel will operate and the in situ quality of the shotcrete are major items concerning USBR engineers when considering the use of shotcrete. The sidewalls of shotcrete-lined reaches of water conveyance tunnels should be well keyed into the rock at the invert level to protect against possible undermining by flowing water. This is particularly important in cases where the bond between excavated surface and shotcrete may not be adequately developed. Figure 19 shows details of shotcrete keyed into the invert at the base of the tunnel sidewall.

106. Detailed studies and cost estimates are employed in establishing definite designs and specifications for a tunnel. Shotcrete often offers economies, chiefly in a partially lined DB tunnel, where it can serve as support and final lining for the tunnel.

107. Shotcrete has been applied in USBR tunnels with steel rib supports where the excavated rock surface required protection to avoid deterioration and sloughing. Usually the thickness applied for this purpose has been 1-1/2 in. (3.8 cm). When this thickness of shotcrete is applied for the full length of a tunnel (to cover all surfaces except the invert), the cost in recent years has approximated 5 percent of the total tunnel cost. If the cost of supporting a tunnel throughout its length with steel ribs is taken at 20 percent of the total tunnel cost, it can be seen that increasing the 1-1/2-in. (3.8-cm) average shotcrete thickness to the thickness required for overall support of the tunnel may be more economical than using ribs supplemented by a layer of protective shotcrete.

108. Figure 18 shows details of a shotcrete "dental work" type of treatment for tunnel surfaces. This type of repair is specified for

seams of deteriorated rock or gouge material that may erode, slake, or expand when exposed to air or water. Seams of varying width and orientation are sometimes encountered in reaches of otherwise competent, erosion-resistant rock. Minimum treatment consists of excavating the soft seam material to a depth from the tunnel surface equal to twice the width of the seam or 6 in. (15.2 cm) maximum. The excavated space is then refilled with shotcrete; the repair material is extended an additional 3 in. (7.6 cm) inside the excavated surface, and is lapped over the excavated surface about 6 in. (15.2 cm) on each side of the seam.

109. Although USBR design drawings and specifications generally describe minimum thicknesses of shotcrete to be applied on an excavated surface, the actual thickness applied depends on the rock conditions encountered. The narrative paragraph "Shotcrete for Tunnel Support" used in recent specifications states: "The areas to be covered and the thickness of the shotcrete shall be as hereinafter specified, as shown on the drawings, or as directed by the contracting officer. Greater thicknesses than those shown on the drawings may be directed or approved by the contracting officer." This approach reflects the opinion of most USBR designers that shotcrete specifications should be flexible enough to permit the field engineer or inspector to increase the shotcrete thickness in critical or potentially critical areas. Periodicals (References 24 and 25) are perused regularly to learn of new information on the design and application of shotcrete.

Design parameters
and rules of thumb

110. Judgment and experience of both E&R Center and construction office personnel are the main guides used in preparing specifications for USBR shotcrete applications. Although some effort has been directed toward the use of the finite element method of analyses for tunnel sections, standard design procedures have not been published. However, mathematical analyses described by Sattler (Reference 26), Mason (Reference 27), Heuer (Reference 19), Deere et al. (Reference 28), and Rabcewicz and Golser (Reference 14) are considered in arriving at a shotcrete thickness for a particular underground opening. The following parameters and

other variables are considered in preparing designs for underground shotcrete applications:

- a. Function of tunnel, shaft, or chamber.
 - (1) Access
 - (2) Water conveyance
 - (a) Pressure
 - (b) Gravity
 - (3) Equipment to be installed
- b. Type of excavation.
 - (1) Machine bore
 - (2) Drill and blast
 - (3) Machine spade
- c. Size and shape (geometry of the opening).
 - (1) Space to accommodate water flow
 - (2) Space to accommodate excavating and shotcreting equipment
 - (3) Space to accommodate maintenance equipment
- d. Geologic factors.
 - (1) Standup time of rock
 - (2) Susceptibility of rock to deterioration by air or water
 - (3) Shear and fault zones
 - (4) Orientation and shape of rock blocks
 - (5) Rock quality (RQD*) and degree of weathering
 - (6) Orientation, continuity, and spacing of major joint sets
 - (7) Shear strength along joint surfaces
 - (8) Amount of water inflow
- e. Shotcrete properties.

* RQD is a modified core recovery classification developed by D. U. Deere and others at the University of Illinois. This classification is based on counting only those pieces longer than 4 in. (10.2 cm) of sound rock in the NX core (Reference 28). Shorter lengths of core are not considered. The percentage is the ratio of total length of core 4 in. (10.2 cm) or longer to the length of the drill hole.

- (1) Compressive strength
- (2) Tensile strength
- (3) Shear strength
- (4) Bond strength between shotcrete and adjacent rock
- f. Flow and abrasion resistance (in water conveyance tunnels).

111. A preliminary guide for the selection of the most commonly used type of primary tunnel supports is provided in the following tabulation:

Support System for Hard-Rock Tunnels	Rock Quality			
	Good (Intact)	Fair (Massive and Moderately Jointed)	Poor (Blocky and Seamy)	Very Poor (Crushed)
Rockbolts	Yes	Yes	?	No
Shotcrete	Yes	Yes	Yes	?
Steel rib supports	Yes	Yes	Yes	Yes

112. In addition to providing alternate bid schedules for either MB or DB excavation, USBR tunnel specifications often provide alternate bidding schedules for types of support systems. For example, Schedule 2 in Cunningham Tunnel Specifications No. DC-7024 required DB construction, steel ribs for primary support, and concrete lining over ribs; Schedule 1 required DB construction and shotcrete for primary support and lining; and Schedule 3 required MB construction and rockbolts for primary support. The low bidder elected to construct the tunnel under Schedule 1.

113. Heuer (Reference 19) developed a "Rule of Thumb" for use in determining the location and thickness of shotcrete for temporary support in 15- to 20-ft-(4.6- to 6.1-m-)diam tunnels. His concept has been expanded by USBR designers to include suggested final support applications in smaller tunnels (Table 2).

114. Where shotcrete is used as a protective coating over excavated surfaces that deteriorate and slough when exposed to air or water, the minimum thickness specified is now 1-1/2 in. (3.8 cm). A recent contract where specifications allowed shotcrete for this purpose was in

Table 2

Typical Shotcrete Applications in Tunnels Experiencing Loosening Loads

Ground Condition	Temporary Support for 15- to 20-ft-(4.6- to 6.1-m-)Diam Tunnels	Final Support for 7- to 15-ft-(2.1- to 4.6-m-) Diam Tunnels
<u>Good Ground</u>		
Where support problems are minimal, but for one reason or another a "bald-headed" tunnel is considered unacceptable. RQD commonly 75 percent or more	2-in. (5.1-cm) thickness in arch above springline	2-in. (5.1-cm) minimum thickness at base of sidewalls, increasing uniformly to 3-in. (7.6-cm) thickness at top of arch
<u>Fair Ground</u>		
Where the rock is more closely jointed or broken and the arch definitely needs some support, but sidewalls are stable. Ground allows good bond of shotcrete to excavation perimeter. RQD commonly 50 to 75 percent	±3-in. (7.6-cm) thickness in arch, feather out below springline	2-in. (5.1-cm) minimum thickness at base of sidewalls, increasing uniformly to 5-in. (12.7-cm) thickness at top of arch
<u>Poor Ground</u>		
Where tunnel walls tend to ravel some, or where a good shotcrete bond to rock cannot be achieved. RQD commonly 25 to 50 percent	3- to 4-in. (7.6- to 10.2-cm) thickness in arch. ±3-in. (7.6-cm) thickness on walls. Carry to base of walls in horseshoe tunnels. On circular tunnels, wall coverage required depends on quality of shotcrete bond to rock. If bond is of fair quality, may need only upper 270-deg shotcrete coverage. If bond is very poor, need full 360-deg coverage	3-in. (7.6-cm) minimum thickness at base of sidewalls, increasing uniformly to 6-in. (15.2-cm) thickness at top of arch. 4-in. (10.2-cm) thickness on invert. Consider use of wire fabric above springline
<u>Very Poor Ground</u>		
If support problems are truly loosening loads, treat as above for poor ground, but add another inch of shotcrete giving 4 to 5 in. (10.2 to 12.7 cm) in arch, and ±4 in. (10.2 cm) on walls		Do not use shotcrete. Support with structural steel ribs and line with concrete
If the ground contains a significant percentage of clay materials, as in joint coatings, is very closely jointed or perhaps crushed, and is at a significant depth below surface so that in situ stresses are high, support problems are probably due to overstraining or combined overstraining and loosening, and ground should be treated as described for overstressed ground (Reference 19)		

the 17-ft 6-in. (5.3-m) horseshoe-shaped Tunnel No. 4 on the Navajo Indian Irrigation Project in New Mexico. This tunnel was constructed under Specifications No. DC-6983. These specifications contain the standard USBR narrative paragraph, "Shotcrete for Protective Coatings." A permanent concrete lining was later placed over the shotcrete coating.

115. Although reliable equipment for applying either dry-mix or wet-mix shotcrete is commercially available, until 1973 all shotcrete applications on USBR underground work specified use of the dry-mix process. However, present thinking is that as long as compressive and bond strengths of the in situ shotcrete are within specified limits at the designated age, shotcrete is adequate regardless of the method of application. The option of using either the dry- or wet-mix shotcrete process was included in the Cunningham Tunnel Specifications No. DC-7024, but the successful bidder chose to use the dry-mix process. The wet-mix shotcreting process was used in the Auburn Dam foundation tunnels in California.

116. USBR provisions for payment for shotcrete are based on the volume of material passed through the shotcrete machine nozzle. Since the awarding of the Cunningham Tunnel contract, many discussions have occurred concerning whether this method of payment favors the use of the dry-mix process. Manufacturers of wet-mix process equipment maintain that the present method of payment for shotcrete is unfavorable to them since applications using the dry-mix process yield more rebound than those of the wet-mix process and, hence, more material for payment. Although the method of paying for shotcrete material through the nozzle is still used by the USBR, consideration is presently being given to other possibly more equitable provisions for payment.

Problem areas in design

117. To design shotcrete sections, the engineer should have data on the behavior of the rock through which an opening is made. These data should include an indication of the pressure that will be exerted on the shotcrete lining and the inward or outward movements of the excavated opening. These data may be obtained from commercially available instruments installed in the excavated opening. This equipment is costly

to purchase, install, and monitor. Also, the instrumentation readings obtained from a particular facility are unavailable to the designer for many months after construction specifications have been issued. Therefore, there is a need for more performance data from rock types that may be encountered in future tunnel work.

118. In their paper, Austin and Fabry (Reference 29) described the pressure cell and extensometer instrumentation installed in Hunter Tunnel in 1972 and the measurements obtained from these devices. For the three locations measured, four out of the five pressure cells installed indicated a divergence of the excavated opening after each heading advance. Similar data from the most commonly encountered rock types are being assembled to assist in future designs. If performance data from the rock formations to be encountered in a particular situation are not available, the designer must rely on experience and judgment.

119. Another problem encountered by designers is that it is seldom possible to apply primary support shotcrete on the invert of DB tunnels. This operation is difficult because the invert is rarely cleaned completely and the railway track is not removed until at a time near the end of the contract. Therefore, it is seldom possible to base a design on a complete ring-type analysis of the section.

120. Few head-loss measurements have been made in shotcrete-lined water conveyance tunnels. Additional prototype hydraulic test data are needed to verify values of roughness coefficient used for flow over shotcrete surfaces. Figures 18 and 19 show the values of roughness coefficient "n" used in Manning's formula for determining the hydraulic properties for rock surfaces anticipated in Cunningham Tunnel.

121. In situ shotcrete variability is a problem to designers. Although 28-day compressive strengths as high as 5000 psi (34.5 MPa) have been specified in large underground openings by other agencies, USBR experience in relatively small tunnels (up to 13 ft (4.0 m)) indicated that considerably lower strengths were obtained. In Hunter Tunnel (1970-73), the specifications called for an 8-hr strength of 500 psi (3.4 MPa) and a 28-day strength of 3750 psi (25.9 MPa) for primary support shotcrete. Most test cores showed lower strengths. The lower

strengths obtained in Hunter Tunnel were generally attributed to (a) difficulties caused by the severely restricted working space, i.e., poor visibility, rebound, fog, dust; and (b) difficulty in getting undisturbed core samples. Since the specified strengths were not consistently obtained in Hunter Tunnel, compressive strength requirements for Cunningham Tunnel (1974) were reduced to 500 psi (3.4 MPa) and 3250 psi (22.4 MPa) at 8 hr and 28 days, respectively. Since compressive strengths for Hunter Tunnel shotcrete were consistently low, field personnel often directed applications of slightly thicker layers of shotcrete than the minimums shown on the specifications drawings.

122. USBR partially lined water conveyance tunnel specifications requiring shotcrete for primary support state generally that "shotcrete shall be applied where directed or approved by the contracting officer and greater thicknesses than those shown on the drawings may be directed or approved by the contracting officer." This statement actually leaves the decision on amounts and locations of shotcrete applications to the tunnel inspector. Where considerable amounts of shotcrete are required, USBR construction engineers have usually tried to utilize their most competent inspection personnel to make decisions on locations and amounts of shotcrete. Wherever possible, an engineer examines the exposed rock immediately after each heading advance. Because of the amount of judgment involved in the tunnel on where and how much shotcrete should be applied and the fact that the shotcrete support varies in thickness due to the unevenness of excavated surface, designers usually provide a larger factor of safety than for concrete facilities. USBR designers generally use a minimum safety factor of 2 for shotcrete designs.

Construction Considerations and Experiences

Equipment requirements and uses

123. The USBR requires that equipment used for mixing and applying shotcrete shall be capable of handling and applying shotcrete containing a specified maximum size of coarse aggregate and an accelerating-hardening admixture. The equipment, including mixers, hoses, nozzles,

nozzle liners, air and water pressure gages, and gaskets, is to be maintained in a clean and proper operating condition. For the dry process, the discharge nozzle must be equipped with a manually operated water injection system having sufficient pressure to provide an even distribution of water into the sand-, coarse-aggregate-, cement-admixture mix. The water valve must be convenient to the nozzleman and be capable of ready adjustment to vary the quantity of water delivered to the mixture. A properly operated air compressor capable of producing clean, dry air adequate for maintaining a uniform nozzle velocity for the application is required.

124. Descriptions of the equipment used at some USBR projects to batch, mix, and apply shotcrete are as follows:

- a. Charles H. Boustead Tunnel, Fryingpan-Arkansas Project. This tunnel, located near Leadville, is a 10-ft- (3.1-m) diam water gravity-flow diversion tunnel approximately 5 miles (8.1 km) in length. Excavation was accomplished by conventional DB methods. Saturated, squeezing ground was encountered approximately halfway through the length of the tunnel. The principal support system used in the tunnel was structural steel ribs. Shotcrete was used in a broad shear zone of inhomogeneous and highly altered rock to prevent sloughing and fallout and as a supplement to the steel supports. Concrete was later placed over the shotcrete for final lining. A double chamber shotcrete machine was used. Aggregate and sand were batched at the tunnel portal using a Winslow Binan batcher and discharged into 4-cu-yd- (3.1-m³) capacity Moran Agitator cars. The agitator cars were transported by rail to the tunnel heading. The aggregate was discharged from the agitator cars onto a short conveyor belt which fed a one-bag mortar mixer where cement was added and the materials mixed. The mixed materials were dumped from the mixer onto a conveyor belt which fed the shotcrete machine. A v-plow mounted over the conveyor made a furrow in the aggregate and cement mixture where the dry Tricosal accelerator was hand-fed from preweighed sacks. The shotcrete operation was conducted in the summer of 1968 when coarse aggregate shotcrete for underground support was quite new to the USBR. Even though the methods were crude by today's standards, satisfactory results were obtained.
- b. Hunter Tunnel, Fryingpan-Arkansas Project. Hunter Tunnel, located near Aspen, is an 8.5-ft (2.6-m) horseshoe-shaped, gravity-flow, partially lined water

diversion structure. Excavation was performed by conventional DB methods. Where competent rock existed, no supports were installed. In reaches where support was required, shotcrete was the primary support for the first 4.5 miles (7.2 km) of this tunnel. The remainder of the tunnel, approximately 3.5 miles (5.6 km), is to be constructed at a later date. The contractor used an Eimco shotcrete machine with a shop-built 5-cu-yd (3.8-m³) capacity, two-compartment hopper car for delivery of materials to the shotcrete machine. Coarse aggregate and sand were prebatched and mixed in a concrete batch plant located near the tunnel portal. The mixed sand and aggregate were dumped into the large compartment of the hopper car directly from the mixer. The cement was weighed and dumped into the small compartment of the hopper car. This car was then moved into the tunnel and attached to the shotcrete machine, which was also mounted on a car. Materials, including Sigunit accelerator, were fed from the hopper car by electric-powered, rheostat-controlled augers mounted in the bottom of the individual hoppers. Calibration of the feed rate of the augers was difficult because voltage fluctuations caused speed variations. Also, wear on the augers affected the feed rate. A 1-1/8-in.- (2.9-cm-) diam True Gunnall Nozzle was used to apply the 3/8-in. (1.0-cm) maximum size aggregate (MSA) shotcrete, which in most cases occurred immediately after a heading advance. Operating air pressure at the machine ranged from 30 to 70 psi (0.21 to 0.48 MPa) with a pressure of 45 to 50 psi (0.31 to 0.34 MPa) generally being used. The pressure of mixing water ranged from 45 to 110 psi (0.31 to 0.76 MPa) with a water pressure of 100 to 110 psi (0.69 to 0.76 MPa) being generally used. This high water pressure was used to ensure that the water jetting from the water ring would penetrate the flow of material passing through the nozzle, thereby producing a uniform shotcrete mixture prior to and after application. Air and water pressures were monitored at the shotcrete machine. Fifty feet (15.2 m) of feeder hose was generally used.

- c. Nast Tunnel, Fryingpan-Arkansas Project. Nast Tunnel, located near Aspen, is a 10-ft 4-in.- (3.2-m) diam MB, circular-shaped, gravity-flow, partially lined water diversion tunnel. Shotcrete was used for secondary support, for protective coating over support metalwork, and for treatment of erodible rock seams in this 3-mile- (4.8-km) long tunnel. The secondary shotcrete was applied after the tunnel was holed through as treatment and final lining over the deteriorated rock and gouge seams. The contractor used a Reed shotcrete machine

with a shop-built 4-cu-yd- (3.1-m^3) capacity, two-compartment hopper car for delivery of materials to the shotcrete machine. Coarse aggregate and sand were pre-batched and mixed in a concrete batch plant located near the tunnel portal. The mixed sand and 3/8-in. (1.0-cm) MSA were dumped into the large compartment of the hopper car. The cement was weighed and dumped into the small compartment of the hopper car. This car was then pulled into the tunnel and attached to the shotcrete machine, which was also mounted on a car. Materials, including Monoset accelerator, were fed from the hopper car by electric-operated, rheostat-controlled augers mounted in the bottom of the individual hoppers. Calibration of the feed rate of these augers was regularly checked to mitigate the effects of wear. A 2-in.- (5.1-cm) diam Gunitite Nozzle was used for the application of shotcrete. An operating air pressure of 50 psi (0.34 MPa) and a water pressure of 100 psi (0.69 MPa) were generally used.

- d. Cunningham Tunnel, Fryingpan-Arkansas Project. Cunningham Tunnel, located near Aspen, is a 3.0-mile (4.8-km) long, DB, horseshoe-shaped, gravity-flow, partially lined water diversion tunnel presently (1974) under construction. Diameter of the concrete-lined section is 7 ft 6 in. (2.3 m). Shotcrete will be used as the primary tunnel support system in those areas where rock conditions permit and will be applied immediately after a heading advance. The contractor will use an Aliva 600 shotcrete machine with a shop-built 6.25-cu-yd- (4.8-m^3) capacity hopper car for transporting materials into the tunnel. Sand, coarse aggregate, and cement are batched and mixed at the inlet portal. A 4-cu-yd- (3.1-m^3) Smith mixer is used for mixing the materials. The materials are discharged from the mixer into the hopper car and transported by rail into the tunnel. A horizontal conveyor belt mounted under the hopper moves the material from the hopper to the shotcrete machine. A slide gate located at the end of the hopper and above the conveyor belt controls the materials feed. A small hopper equipped with a feed rotor is mounted near the end of the horizontal conveyor belt for dispensing the dry GS-25 accelerator. The rate of accelerator feed is controlled by the speed of the rotor. The accelerator feed rotor is powered by a direct chain from the drive pulley on the horizontal materials conveyor belt. This allows an accurate ratio of accelerator to cement, sand, and coarse aggregate, regardless of the speed of the conveyor. Three-eighths-in. (1.0-cm) MSA is to be used. This shotcreting equipment is pneumatically powered. Experience indicates that in an underground environment the pneumatically powered equipment is superior to

electrically powered equipment. Dust generated by the shotcreting operation fouls the complex electrical systems, causing considerable equipment malfunctions that require skilled technicians to repair. The pneumatically operated equipment is basically mechanical and can usually be repaired by the average miner, thereby reducing costs. Materials are discharged from the gun to the nozzle through a 2-in.- (5.1-cm) inside-diam (ID) hose. The nozzle is fitted with the standard water ring, but an additional water ring has been installed in the delivery hose 10 ft (3.1 m) back of the nozzle. This ensures a more thorough wetting of the mixed ingredients.

- e. Intake tunnel, pumping chamber, and shaft, Southern Nevada Water Project. This water conveyance project is a pressure system located near Henderson, Nevada. Rockbolts, 7 and 12 ft (2.1 and 3.7 m) long, were the primary support system used in these facilities. Shotcrete was applied as a final lining on excavated surfaces to provide protection against erosion of flowing water after the facilities were placed in service, and as a protective coating over the ends of rockbolts, associated wire fabric, and metal pans. Brief descriptions of three areas where shotcrete was used during construction in 1969 and 1970 follow:

- (1) To the arch and sidewalls of the 13-ft- (4.0-m) diam, 750-ft- (229-m) long, horseshoe-shaped, full-flow, partially lined tunnel. Thickness of the shotcrete was 4 to 6 in. (10.2 to 15.2 cm) with 1315 cu yd (1006 m³) used and was applied after the tunnel had been excavated and supported by rockbolts.
- (2) To a vertical access shaft with a rectangular shape of 22 by 10 ft (6.7 by 3.1 m) by 19¹/₄ ft (59.2 m) in depth. Some 463 cu yd (354 m³) of shotcrete were applied to a thickness of 4 in. (10.2 cm) after the shaft was completely excavated.
- (3) To an underground pump chamber which was covered with 375 cu yd (287 m³) applied over 4- by 4-in. (10.2- by 10.2-cm) wire fabric to a thickness of 4 to 6 in. (10.2 to 15.2 cm). The dimensions of the chamber were approximately 75 by 20 by 10 ft (23 by 6 by 3 m). Shotcrete was applied after rockbolts were installed and tightened in the pump chamber.

An Aliva Model 300 pneumatic shotcrete machine was used with the material dry mixed in a concrete transit-mix truck and delivered to the shotcrete machine hopper by a front-end loader. The rock walls were cleaned, and the running and dripping water was collected prior to applying the shotcrete. It was necessary to reclean the rock

electrically powered equipment. Dust generated by the shotcreting operation fouls the complex electrical systems, causing considerable equipment malfunctions that require skilled technicians to repair. The pneumatically operated equipment is basically mechanical and can usually be repaired by the average miner, thereby reducing costs. Materials are discharged from the gun to the nozzle through a 2-in.- (5.1-cm) inside-diam (ID) hose. The nozzle is fitted with the standard water ring, but an additional water ring has been installed in the delivery hose 10 ft (3.1 m) back of the nozzle. This ensures a more thorough wetting of the mixed ingredients.

- e. Intake tunnel, pumping chamber, and shaft, Southern Nevada Water Project. This water conveyance project is a pressure system located near Henderson, Nevada. Rockbolts, 7 and 12 ft (2.1 and 3.7 m) long, were the primary support system used in these facilities. Shotcrete was applied as a final lining on excavated surfaces to provide protection against erosion of flowing water after the facilities were placed in service, and as a protective coating over the ends of rockbolts, associated wire fabric, and metal pans. Brief descriptions of three areas where shotcrete was used during construction in 1969 and 1970 follow:

- (1) To the arch and sidewalls of the 13-ft- (4.0-m) diam, 750-ft- (229-m) long, horseshoe-shaped, full-flow, partially lined tunnel. Thickness of the shotcrete was 4 to 6 in. (10.2 to 15.2 cm) with 1315 cu yd (1006 m³) used and was applied after the tunnel had been excavated and supported by rockbolts.
- (2) To a vertical access shaft with a rectangular shape of 22 by 10 ft (6.7 by 3.1 m) by 19¹/₄ ft (59.2 m) in depth. Some 463 cu yd (354 m³) of shotcrete were applied to a thickness of 4 in. (10.2 cm) after the shaft was completely excavated.
- (3) To an underground pump chamber which was covered with 375 cu yd (287 m³) applied over 4- by 4-in. (10.2- by 10.2-cm) wire fabric to a thickness of 4 to 6 in. (10.2 to 15.2 cm). The dimensions of the chamber were approximately 75 by 20 by 10 ft (23 by 6 by 3 m). Shotcrete was applied after rockbolts were installed and tightened in the pump chamber.

An Aliva Model 300 pneumatic shotcrete machine was used with the material dry mixed in a concrete transit-mix truck and delivered to the shotcrete machine hopper by a front-end loader. The rock walls were cleaned, and the running and dripping water was collected prior to applying the shotcrete. It was necessary to reclean the rock

frequently as cement dust from the operation would coat the surface and prevent adequate bond. Because of the dirty, uncomfortable, and somewhat unhealthy conditions associated with shotcreting and other tunnel work, the labor turnover was high and experienced nozzle-men were scarce. This coupled with the poor visibility while shotcreting and equipment problems very often resulted in improper nozzle use and other difficulties impeding high-quality shotcrete production. Furthermore, the shotcrete operation should have been kept close to the heading, but with other mining equipment in the tunnel, this became impossible at times.

- f. Tunnel No. 3, Navajo Indian Irrigation Project. Shotcrete was used in this 3.5-mile (5.6-km) long, gravity-flow tunnel located 35 miles (56.3 km) southeast of Farmington, which was excavated in 1971 and 1972. Rockbolts were the primary supports in the tunnel. The main purpose of the shotcrete was protection of the weather-sensitive shales and sandstones. Final concrete lining was placed over the shotcrete coating. This 19.5-ft- (5.9-m) diam circular tunnel was excavated with a tunneling machine. Approximately 540 cu yd (413 m³) of shotcrete were used to reduce water seepage in two large fallout areas in the tunnel crown. About 200 cu yd (153 m³) of shotcrete were applied 3 in. (7.6 cm) thick to the walls and crown to protect the weather-sensitive formation. It was estimated that 50 percent rebound occurred during most of this shotcrete application. The shotcrete equipment consisted of a Meynadier AG Type V shotcrete machine made in Zurich, Switzerland, equipped with an air motor and an Acrison additive feeder driven by an electric motor, and a Moyno water pump. Batching consisted of charging transit mixers with 4800 lb (2174 kg) of sand, 2000 lb (906 kg) of 3/4-in. (1.9-cm) aggregate, and 25 gal (95 dm³) of water and then mixing 3 min. The transit mixers then hauled the mixed material to the stockpile. The mixture of sand and aggregate was then removed from the stockpile with a front-end loader and deposited into an agitator car, along with seven bags of Type III cement per cubic yard (390 kg/m³). The cement was added by hand. The car was moved into the tunnel, and the aggregate was fed into the shotcrete machine along with 4 percent "Isocrete" accelerator additive. The mixture was forced to the nozzle by air pressure, with water added at the nozzle. Since the shotcrete machine was not equipped with a gage, the rotational speed was measured with a tachometer and ranged from 600 to 900 rpm. Rebound loss was estimated at 50 percent. Application of the shotcrete immediately behind the cutting head of the TBM was very difficult

because of the minimum clearance between the bore and the equipment.

- g. Tunnel No. 4, Navajo Indian Irrigation Project. Tunnel No. 4, a 1-mile (1.6-km) long, gravity-flow structure, is located approximately 30 miles (48 km) southeast of Farmington, New Mexico. Excavation for this was completed during 1973 and 1974 by conventional DB methods resulting in a 20.5-ft (6.2-m) horseshoe section. Shotcrete 1-1/2 in. (3.8 cm) thick was used in this tunnel for preventing surface deterioration of the air- and water-sensitive shale and sandstone formations. Steel sets were installed as the primary support. Concrete lining was subsequently placed over the shotcrete. A Reed dry-mix shotcrete machine mounted on the back of a transit-mix truck with a Blaw Knox mixer was used. A screw conveyor moved the mixed sand and cement from the mixer to the shotcrete machine. Unicrete accelerator was fed into the screw conveyor at the rate of 1 lb (0.45 kg) per bag of cement. Coarse aggregate was not used in the mix. Rebound loss was estimated to be between 10 and 15 percent. This application of shotcrete was very effective in preventing surface deterioration in the weather-sensitive formations. In addition, it eliminated most of the requirements for lagging behind the steel supports in the walls. Application of shotcrete close to the heading was limited. Shotcrete applied closer than 12 ft (3.7 m) to the heading was occasionally damaged by blasting. However, in the formations encountered in this tunnel it was desirable to apply the shotcrete in the crown closer to the heading and then repair any damaged shotcrete after blasting.
- h. Subsurface excavations - Auburn Dam, Central Valley Project. The specifications for the tunnel work associated with the Auburn Dam foundation contract required shotcrete to be applied by the dry-mix process. Pursuant to a request by the contractor and the attendant benefits to the Bureau, the requirements were waived to permit the use of the wet-mix system. The wet mix was delivered to the jobsite in 8-cu-yd (6.1-m³) Challenge transit mixers and discharged into the hopper of a diesel-powered Thomsen A-7 pump. This shotcrete mix was pumped through a 4-in. (10.2-cm) line to a second diesel-powered Thomsen A-7 pump inside the tunnel, which was equipped with a powder accelerator dispensing unit. Operating as a unit with this pump was an air drier and accelerator supply tank. The pump in the tunnel fed the shotcrete mix to the nozzle where air, containing previously added accelerator, forced the material at high velocity to the tunnel walls. Some difficulties were

encountered with the accelerator dispenser calibration and thus required some special monitoring and other effort to assure quality control. As the job progressed, other mechanical problems developed with the dispenser, ultimately forcing the contractor to change equipment. He obtained a Challenge squeeze-crete shotcreting machine which used powdered accelerator formed into 6-in.-diam by 12-in.-long (15.2- by 30.5-cm) logs. Sigunit and Isocrete accelerators were both tried initially, but the logs used in the second system were made from Sigunit powder. The squeeze-crete pump which replaced the Thomsen pump in the tunnel performed well in the shotcreting operation, although its capacity seemed to be somewhat less than that of the Thomsen.

Materials requirements and experiences

125. Cement used by the USBR in shotcrete is generally Type II and low alkali meeting requirements of Federal Specification SS-C-192g, although some Type III, high early strength cement has been used. In most cases, the cement used has been tested by the Government or accepted on certification that it meets applicable specifications.

126. The usual quality requirements for concrete aggregate also apply to aggregate for use in shotcrete. These requirements are associated with the parameters of specific gravity, soundness, absorption, and gradation. In some instances, an increased amount of material passing the 100-mesh sieve has been allowed, if needed, to provide added workability and adhering qualities of the shotcrete. The sand and coarse aggregates used in shotcrete usually are naturally occurring materials frequently supplemented with minor amounts of crushed materials.

127. Most recent specifications for coarse aggregate shotcrete used for primary support have required the use of an accelerating-hardening admixture which is added to the mix in powder form. The use of calcium chloride has not been allowed. It is recognized that where the shotcrete application is used strictly for sealing the rock against moisture change, high early strength is not usually required, and the use of accelerators may not be warranted.

Shotcrete mixes

128. Although the shotcrete mix used is subject to the approval of

the contracting officer on the specific project, specifications require the contractor to determine the exact proportions of cement, sand, coarse aggregate, and accelerating-hardening admixture. In some instances, specifications have required a minimum of seven bags of cement per cu yd (390 kg/m^3) of shotcrete. Requirements for 8-hr and 28-day compressive strengths will usually control the exact mix proportions. The sand, coarse aggregate, cement, and admixture must be thoroughly mixed before being fed into the delivery equipment. Machine mixing of the dry ingredients, except the accelerating admixture, is required.

129. Some examples of mixes used at projects are shown in the following tabulation:

	<u>Pounds per cu yd (kg/m³)</u>				<u>Admixture, percent by Weight of Cement</u>
	<u>Cement</u>	<u>Sand</u>	<u>Coarse Aggregate</u>		
			<u>3/16 in.</u>	<u>3/8 in.</u>	
			<u>(0.5 cm)</u>	<u>(1.0 cm)</u>	
			<u>to</u>	<u>to</u>	
		<u>3/8 in.</u>	<u>3/4 in.</u>		
		<u>(1.0 cm)</u>	<u>(1.9 cm)</u>		
Southern Nevada	658 (390)	1692 (1003)	738 (438)	345 (204)	4.5 to 8.0
Hunter Tunnel	658 (390)	1749 (1037)	1415 (839)	0	4.0
Nast Tunnel	658 (390)	1574 (933)	1288 (764)	0	3.0 to 4.0
Cunningham Tunnel	658 (390)	1701 (1009)	1427 (846)	0	2.5 to 4.0

130. In most instances, the required compressive strengths have been 500 psi (3.4 MPa) at 8 hr and 3000 to 4000 psi (20.7 to 27.6 MPa) at 28 days. Some difficulty has been experienced in achieving these strengths from the same mix. If the 8-hr requirement is met, then the 28-day strength may fall below the desired strength. It is known that shotcrete accelerators, although promoting high early strength, can cause a reduced strength development at increased age, depending upon the quantity of accelerator used. Also, the temperature of the materials and shotcrete mix can influence the effectiveness of the accelerator.

Application requirements and experiences

131. Since shotcrete is a relatively new material in the construction of underground facilities, and because of the difficult and environmentally unattractive working conditions, experienced craftsmen capable of applying shotcrete and operating support equipment are generally scarce. Nozzleman should demonstrate their ability to apply shotcrete of the required quality prior to application of shotcrete for underground support. Fellow workmen must have confidence in the material that is supporting the ground they are working under. The owner wants a material that will support anticipated loads for the required life of the structure. The builder wants the material applied in a way that minimizes his cost in the application. The USBR requires the nozzleman to shoot test panels to demonstrate his ability to apply the shotcrete and the ability of the application and batching equipment to deliver the proper ingredients to the ground requiring support. These panels also will provide information on the mix design.

132. Shotcrete application underground, particularly in the long small-diameter tunnels built by the USBR for water conveyance, is accomplished in cramped quarters under less than desirable conditions. Adequate lighting is required so that the surface to be covered can be seen by the nozzleman. Surfaces to be coated must be clean and free of running or dripping water. It is necessary to collect the water inflow in pipes prior to shotcrete application. Dry cement dust from previous applications must be removed from rock surfaces to assure a good bond.

133. When a leading shield is used for TBM support, it is imperative that the shotcrete be applied immediately behind the shield. If shotcrete is applied on the sidewalls ahead of the thrust grippers of the TBM, it is usually necessary to repair these areas after the grippers have moved on.

Safety and environmental requirements

134. The contractor is required to maintain safety in all areas where shotcrete is being applied. Sodium and/or potassium hydroxide and

possibly other chemicals contained in accelerating-hardening admixtures are moderately toxic and, when used in shotcrete, may cause skin and respiratory irritation unless adequate safety measures are taken. To assure adequate protection against toxic materials and dust, nozzle men and helpers are required, when handling and applying shotcrete containing an accelerating-hardening admixture, to wear gloves, adequate protective clothing, and sand-blasting hoods supplied with filtered air that is free of objectionable or toxic material. U. S. Bureau of Mines-approved respirators equipped with a chemical filter that will prevent passage of caustic mists may be used in lieu of hoods provided goggles or safety glasses are worn. Protective ointments for use by workmen to minimize skin irritation are to be readily available near the shotcreting operation. Air hoses are required to be equipped with safety-type couplings and secured with wire or chain at each coupling to prevent whipping in the event of a failure.

135. Although shotcreting takes place inside of a tunnel, the environmental effects can be felt outside as well. The chemical accelerator and portland cement included in the shotcrete are also contained in the rebound material which ends up on the invert of the tunnel. These chemicals are leached from the wasted shotcreting material and carried by the water flow out of the tunnel into downstream tributaries. To preclude these and other pollutants associated with the tunnel construction from entering downstream water sources, the USBR requires the contractor to furnish and operate a water treatment plant.

Construction difficulties and problem areas

136. Many of the difficulties which are encountered in the Bureau's use of shotcrete have already been mentioned in other areas of this report. However, it seems well to reiterate some of them as well as others here.

137. Very often when a contractor is starting a tunnel and preparing his equipment he seemingly forgets that there will be a 28-day delay between the time he shoots his test panels and the time the Bureau approves the mix design and strengths. Furthermore, there is a tendency

for contractors to shoot test panels for one mix design, accelerator content, etc., rather than from several mixes, thereby assuring an approved mix. Another situation which arises is that the nozzleman who shoots the test panels and demonstrates his capabilities may not remain on the job for the contractor's total tunnel shotcrete application.

138. Space limitations adjacent to equipment in the Bureau's long, small-bore tunnels present many different construction difficulties relative to the application of shotcrete near the head. Shotcrete application is hampered by the typically poor lighting, fog dust, etc., from shooting, access, and other factors. Furthermore, timely inspection of the in-place shotcrete is also hampered by the same factors. More and improved use of remote shotcreting could simplify some of the difficulties. Increased use of the wet system of shotcreting could improve the environmental problem. The availability and use of improved low-caustic accelerators that become environmentally inert a few hours after application would be an improvement.

Testing and Evaluating Considerations and Experiences

139. Shotcrete testing during construction differs considerably from concrete testing. The term 3000-psi (20.7-MPa) concrete means that an average value of 3000 psi (20.7 MPa) was obtained by breaking 6- by 12-in. (15.2- by 30.5-cm) control cylinders in compression at a certain age. In the industry, the 6- by 12-in. (15.2- by 30.5-cm) cylinder has become the standard specimen not because it necessarily represents the property of the in-place material, but because so much experience has been amassed with it that confidence in designs can be based on its test results. When reference is made to 3000-psi (20.7-MPa) shotcrete, the full meaning is ambiguous. The reference may be to compressive strengths of 3- or 6-in. (7.6- or 15.2-cm) cubes or of 2- or 4-in. (5.1- or 10.2-cm) cores. In shotcrete quality control, there is presently no standard specimen size to which all compressive strength results are related. It is, of course, possible to fabricate 6- by 12-in. (15.2- by 30.5-cm) cylinders by shooting into wire mesh cylinders, but such specimens probably

differ considerably from the in-place material. Each test method or specimen size gives different results, some varying by 20 percent or more. No test methods give the exact strength of the in-place material. It is therefore important to report not only the test results but also the size and shape of the test specimen used.

140. Compressive strength is the most widely used quality-control parameter for shotcrete and the one generally used by the USBR. There are many factors that affect shotcrete quality, such as good mix design and batch plant control; good workmanship and equipment maintenance are particularly important. Visual inspection during and after application can detect variations in water content, workmanship deficiencies, and sand-pocket inclusions or laminations. Visual inspection can also detect gross variations in accelerator content. The final quality evaluation in USBR shotcrete work, however, is compressive strength.

141. Since failures, when they do occur, usually result from combinations of compressive, shear, and tensile overstresses, the validity of evaluating shotcrete primarily on the basis of compressive strength may be questioned. In some cases, abrasion resistance and permeability may be very important factors in the quality desired. There are, however, some good reasons for relying so heavily upon the compressive strength test for shotcrete quality. Generally, higher compressive strength is accompanied by higher abrasion resistance, lower permeability, higher shear strength, and higher tensile strength. Compressive strength test results usually show less scatter and are therefore easier to evaluate than tensile strength results; it is difficult to design a shear test that does not also apply a bending moment to the specimen. Finally, compressive strength tests are more convenient to perform; they take less time, skill, and specialized equipment than abrasion, permeability, shear, or tensile tests. Unless the uniqueness of the problem warrants such specialized testing, compressive strength testing is usually sufficient.

USBR practices and experiences

142. USBR specifications for tunnel support shotcrete normally require minimum average compressive strength values at 8 hr and at

28 days. Typical requirements are 500 psi (3.4 MPa) at 8 hr and 3250 psi (22.4 MPa) at 28 days based on tests of NX-size cores having a length-diameter ratio of 2. The 8-hr requirement is necessary to ensure that the set accelerating admixture is producing sufficient early strengths to resist the shock of blasting and rock movement. The 28-day strength requirement is necessary to ensure structural integrity and long-term durability.

143. At least 30 days prior to application of shotcrete, the contractor is required to fabricate test panels for each mix to be used. This may or may not be at the construction site, but the same equipment and materials to be used during construction must be used during fabrication of the panels, and a Government inspector must be present. The primary purpose of the preconstruction testing is to confirm that the mixes and equipment used are capable of producing shotcrete of the specified strengths and quality. It is to the contractor's advantage to try several mixes including several brands and percentages of accelerator to avoid the delay that may result if only one mix is tested and if it fails to meet the 28-day strength requirement. As mentioned in paragraph 131, the nozzleman who fabricates the test panels should be the same one who applies the shotcrete in the tunnel so that his capabilities and procedures can be evaluated. If the 8-hr requirement is not met, the delay is minimal since more mixes can be tested the following day, but failure to meet the 28-day requirement results in at least a 28-day delay in testing additional mixes. The preconstruction testing also serves, in the absence of a standard acceptance test, to evaluate the cement-accelerator compatibility.

144. Preconstruction test panels must be at least 18 in. (45.7 cm) square with two or more sides open or with tapered edges to prevent rebound inclusion. Two or more vertical and horizontal panels are fabricated for each mix. USBR laboratory studies show that about 1000-psi (6.9-MPa) compressive strength is required to obtain good 2-in.- (5.1-cm-) diam cores from concrete or shotcrete. Therefore, sawed cubes are usually used to obtain 8-hr compressive strengths, which are normally less than 1000 psi (6.9 MPa). Four-in. (10.2-cm) cubes are tested

at 8 hr, and either 4-in. (10.2-cm) cubes or NX-size (2-1/8-in. (5.4-cm) diameter) cores are tested at 28 days. Strengths obtained on 4-in. (10.2-cm) cubes are multiplied by 0.86 to convert to core strength. Results of tests on cores having length-diameter ratios other than 2 are adjusted to the standard 2-L/D ratio. Panels and specimens are protected from moisture loss by polyethylene sheeting or other adequate means until the time of testing.

145. Preconstruction testing does not guarantee that good quality shotcrete will be applied in the tunnel. The contractor generally uses extreme care and his best personnel in fabricating the panels, and working conditions are usually much better than during construction. Preconstruction testing does demonstrate that the mix design and the equipment are capable of producing shotcrete of the specified strengths. Once a mix is established, it is not to be changed without prior USBR approval. In some cases, this approval will only be given after further testing of panels to ensure that the changed mix will meet specifications requirements. The percent and type of accelerator used in the successful test panels must be used throughout the job unless USBR approval for change is given. USBR specifications seldom require 8-hr strength tests to be made during construction, so it is essential that the prior approved mixes remain unchanged.

146. Tests are required from the in-place shotcrete at 28 days. Diamond-drilled NX-size cores are extracted from the tunnel lining by the contractor. Cores with L/D ratios of 2 are obtained whenever possible. Typically, three cores suitable for testing are to be extracted for each 100 (linear) ft (30.5 m) of shotcrete placed in the tunnel. If, as the work progresses, the compressive strengths of the test cores are satisfactory, the frequency of testing may be reduced. If the compressive strengths are not satisfactory, more testing may be required, and a careful examination of the entire shotcrete operation from batch plant to the equipment used, including nozzleman technique, is conducted.

147. Extracting cores from the lining may appear to be the ideal method of testing the in-place material. It is probably the most representative test method in common use, but it does have shortcomings. The

time lag associated with testing cores at 28 days can result in poor quality shotcrete being applied in the interim. Furthermore, the use of worn drilling equipment and improper drilling procedures increases the chances of damage to cores with attendant adverse effects on test results.

148. When strength difficulties were encountered in the USBR's Hunter Tunnel, an attempt was made to determine if the test method may have contributed to the problem. In August 1972, a shotcrete slab approximately 18 by 18 by 6 in. (45.7 by 45.7 by 15.2 cm) was sawed from the lining of Hunter Tunnel and delivered immediately to the USBR laboratories. This slab was cut to include the core holes where the contractor had previously extracted cores for testing at 28 days. Three of these NX-size cores were sent to Denver with the slab. In Denver, NX cores, 3-in. (7.6-cm) cubes, and 4-in. (10.2-cm) cubes were cut from the slab and tested in compression as were the cores that had been extracted by the contractor before the slab was removed from the lining. All tests were conducted 28 days after the shotcrete was applied. The drilling technique apparently had an effect on the strengths obtained. A slight curvature was noted in the cores drilled in the tunnel. The average strength of cores drilled under difficult conditions in the tunnel was about 10 percent lower than the average strength of cores drilled under ideal laboratory conditions. Tests on cubes sawed in Denver indicated that the use of NX-size cores in itself was not causing low compressive strengths. Regardless of size, none of the tested specimens had strengths meeting the specifications requirement of 3750 psi (25.9 MPa) at 28 days. The primary problem was apparently not in the test method but in quality control of the application when working under adverse conditions. In some subsequent specifications, the 28-day compressive strength requirement for the shotcrete has been reduced to 3250 psi (22.4 MPa) as discussed in paragraph 121. Although strengths of 3750 psi (25.9 MPa) and higher can be obtained even under difficult conditions, such requirements are not always practical.

149. Another qualitative test, not used for specifications compliance, but useful for comparative testing, is made using the impact or rebound hammer. This testing technique is normally used only when a

problem with some apparently low-quality shotcrete is encountered and a quick survey of relative strengths is desired. A series of readings is taken at each location to be evaluated; then they are averaged and compared.

Research and test
methods for the future

150. Several ways to improve the testing of shotcrete are now under investigation by the USBR. One of these is a pullout method. A bolt with a washer brazed to the head is positioned so that it will be embedded in the fresh shotcrete when it is applied. At the time of testing, this bolt is pulled from the shotcrete through a metal ring, and a truncated cone of shotcrete is extracted. The force required to extract the cone is divided by the fractured surface area of the cone to obtain a pullout-failure stress. This arbitrary value is then converted to conventional test specimen values, using correlation curves established previously in the laboratory. This test method could possibly provide a means for testing the in-place strength of the shotcrete lining before it has attained the strength necessary for successful drilling of cores.

151. Another study is under way to generate data which can be used in the interpretation of shotcrete core and cube test results. The USBR has tested over a thousand specimens in an effort to define the relationships between compressive strength values obtained from tests on various sizes of sawed cubes and drilled cores. An attempt will be made to determine whether larger diameter cores give less variable results. If they do, we may at some future time specify cores with the length equal to the diameter so that large-diameter specimens can be obtained from thin linings.

152. There is no doubt that testing methods for shotcrete will be improved in the future. For the present, however, the USBR relies mainly on cores drilled from the lining and on cores or cubes cut from panels with the size of specimens and relationship between them stated in the specifications.

Summary and General Discussion

153. Shotcrete containing accelerators and either coarse and/or fine aggregates has been used successfully for some time by the USBR in underground water conveyance facilities. These uses have been primarily for protection of excavated surfaces and temporary support. Shotcrete used for primary tunnel support is still a relatively new tool with the USBR. Although some experience has been gained in application, a long-time history of performance in service is not available. Prior to 1974, the USBR had used only dry-mix process shotcreting for tunnel support with the accelerator being added as a powder. The wet-mix shotcreting process is presently (1974) being used at Auburn Dam as permanent replacement for temporary tunnel supports in foundation exploration tunnels.

154. Difficulty was experienced at Hunter Tunnel in Colorado in obtaining specified compressive strengths. Some lining and rock fallouts have occurred at this tunnel, but it has not been established whether this was the result of ground conditions or inadequate support from the shotcrete application, or a combination of both.

155. Experience gained while shooting test panels at Cunningham Tunnel in 1974 indicated that the skill of the operator was a major factor in the quality of the shotcrete. Skilled operators were able to obtain up to three times the compressive strengths on test specimens as those obtained by unskilled operators using the same shotcrete mix. Poor visibility, resulting from dust and water vapor, and inadequate lighting, frequently encountered in tunnels, compound the difficulties of quality control in shotcrete applications. Because application procedures play such an important part in the final quality of the shotcrete, USBR engineers are very aware that specimens taken from test panels made under favorable conditions do not adequately represent the quality of the in situ shotcrete placed under adverse conditions. Cores have been drilled from the in situ linings to check the shotcrete quality. In the case of thin shotcrete linings, it is difficult to maintain L/D ratios of 2 because core diameters become too small for reliable and consistent

results. The pullout test method and the use of cores with L/D ratios of 1 are being explored as alternate means of measuring in-place quality and uniformity.

156. The use of the wet-mix process shotcrete in exploratory tunnels at Auburn Dam has only recently been done, and an overall evaluation of the merits of this system relative to the dry-mix system has yet to be made. The wet-mix process does not seem to have the potential for as high a strength as the dry-mix process. Nevertheless, the requirements of 500 psi (3.4 MPa) at 8 hr and 3000 psi (20.7 MPa) at 28 days have been obtained with both processes. Better visibility during shotcreting because of reduced dust and water vapor in the atmosphere aids in obtaining a more uniform application and consistent quality. Difficulty was experienced with gumming of the accelerator feeder because of the high humidity in the tunnel, and some consideration was given to the use of liquid accelerator. The accelerator was introduced along with the air stream. No detailed evaluation was made of the uniformity of dispersement of the accelerator throughout the shotcrete.

157. Although aggregate sizes up to 3/4 in. (1.9 cm) have been used, the most commonly used maximum size is 3/8 in. (1.0 cm). The percent of the aggregate which is sand has ranged from 50 to 65 percent. Fifty-five percent sand in shotcrete containing 3/8-in. (1.0-cm) MSA produces a good shotcrete mix with most aggregates. At C. H. Boustead Tunnel, the percent of sand which would pass a No. 100 screen was increased to 14 percent. The increased fines did not appear to improve the application or the quality of the shotcrete.

158. The expanded use of shotcrete in USBR underground construction has brought with it additional problems and unanswered questions. Although the USBR has a continuing research endeavor in this area, adequate funds are rarely available to pursue the solution of problems at the pace desired. Some areas where further studies seem warranted are:

- a. Product variability. Ways need to be found for reducing product variability from the standpoint of materials and workmanship. Equipment, personnel qualifications, and quality control testing all play a part in this situation.

- b. Roughness coefficients. Additional prototype hydraulic test data are needed to verify values of roughness coefficient used for flow over shotcrete surfaces.
- c. Application of shotcrete in invert. A difficult problem, but one needing a solution, is how to apply shotcrete to the invert of a DB tunnel with the rails, pipes, rebound material, and other obstacles hindering application.
- d. Remote equipment. Develop new equipment or refine existing equipment for remotely applying shotcrete near the head of the tunnel.
- e. Accelerators. Develop new or evaluate existing low-caustic, nonhazardous additives for accelerating the set time of the shotcrete.
- f. Quality control. Further developments in in-place testing of shotcrete can help industry establish simplified quality control test procedures, possibly by using the pullout method.

PART IV: SHOTCRETE INSTRUMENTATION

by

R. E. Mason*

Introduction

159. The use of coarse-aggregate shotcrete for underground support was developed in Sweden and Austria in the mid-1950's. In Sweden, shotcrete was generally used as a thin protective coating to prevent the development of loosening pressures in hard fractured rock. The design of such a lining was dictated by experience and rule of thumb. On the other hand, early work in Austria was carried out in soft or badly fractured ground, utilizing a structural thickness of shotcrete reinforced with wire mesh and steel truss arches. The theoretical basis of this design provided the incentive to investigate loading conditions and resulting lining displacements by means of instrumentation. The method of construction developed in Austria, using a theoretical-observational approach to design, was called the New Austrian Tunneling Method (NATM).

160. The NATM is a method of excavation and support whose aim is to provide a rapidly placed flexible support system, which reduces loosening pressures to a minimum but allows pressures due to plastic flow to dissipate in deformation. The support system generally includes a 10- to 15-cm thickness of shotcrete, optionally reinforced with wire mesh and lightweight U-shaped steel ribs or fabricated steel truss arches. Grouted rock anchors of the SN-type are added in poorer ground conditions to build a "rock arch" about the tunnel section. The supporting function of this "rock arch" becomes much greater than that of the reinforced shotcrete lining in large tunnels or where extreme plastic deformations occur. The steel ribs vary in weight from approximately 12 to 26 lb/ft (linear) and are often deformed in the field to conform more closely to the shotcreted rock surface. Thus,

* Senior engineer for A. A. Mathews, Inc., Rockville, Maryland.

their arch shape and alignment are lost and they act simply as reinforcement rather than as conventional steel ribs. The ribs are fastened to the rock with anchors and buried with shotcrete. Conventional methods of advance in poor ground conditions, such as multiple drifts and forepoling or spiling, have been successfully incorporated into the procedure.

161. Design of the support system and excavation sequence is by a theoretical-empirical approach. From a plastic design analysis and past experience, the system is designed and dimensioned, and instrumentation is extensively used to indicate necessary changes as the tunnel proceeds. The NATM aims to achieve a factor of safety of as close to unity as possible.

162. Coarse-aggregate shotcrete for underground support was introduced to this continent in 1967 at the Canadian National Railroad (CNR) Vancouver Tunnel and has since become a widely used construction material. A most notable current example of its use is in the construction of the Washington Metro System. Shotcrete, combined with rockbolts and steel ribs, provides the temporary and permanent support for subway tunnels and stations excavated in rock. Extensive instrumentation of rock displacements and lining strains has contributed greatly to the acceptance and understanding of this type of support.

163. Instrumentation of shotcrete-lined tunnels currently includes devices to measure stresses, strains, or pressures in the shotcrete lining; rock mass movements relative to individual borehole anchors; and lining deformation or closure. Stressmeters, strainmeters, or pressure cells are utilized to monitor the effects of loading in the lining, and pressure cells can also be oriented to monitor pressures between the lining and the ground. Borehole extensometers measure relative displacement between individual anchors. Multiple-position extensometers have up to eight anchors which may be set at any desired depth in the borehole. Closure is generally measured horizontally between two fixed points in the arch or opposing sidewalls, but diagonal and vertical measurements are also made.

164. All three methods of measurement are utilized in current

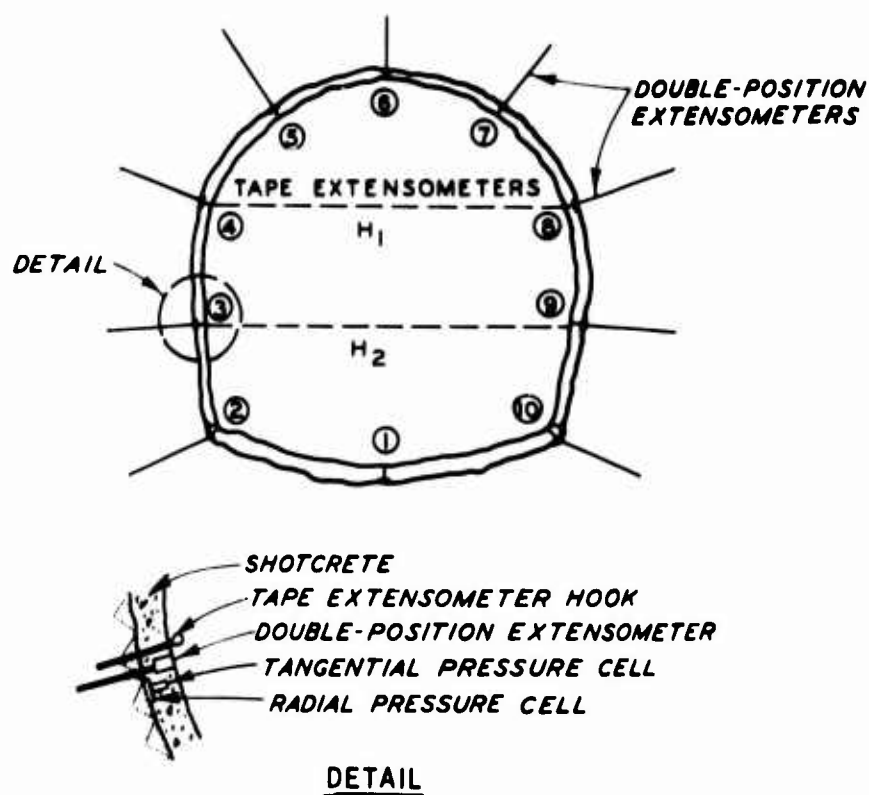


Figure 20. Typical measurement station for the NATM

practice of the NATM. Figure 20 shows a typical measurement station for a tunnel driven by the NATM.

Gloetzl Pressure Cells

165. The Gloetzl hydraulic pressure cells were developed for the measurement of stresses in concrete tunnel linings and between the lining and the rock or soil. The cells have a thin, strong sensing pad which is embedded within the concrete lining or between the lining and the surrounding ground. The stress or load over the pad area from the surrounding material applies pressure to the pad, which in turn is transmitted to fluid within the pad. A compensating diaphragm relief valve is connected to the pad by a short length of high-pressure tubing through which the fluid pressure is transmitted. Fluid is delivered to the other side of the diaphragm through high-pressure tubing from a

hydraulic pump. When the pressure at the pump exceeds the pressure on the pad side of the valve by 1.5 psi, the diaphragm valve opens and the fluid is allowed to return to the pump through the return line. The pressure at which the valve opens, as read on the gauge at the pump, minus 1.5 psi, is approximately equal to the static pressure inside the pad. This approximation is further improved if the volume rate pumped is very small at the moment of valve opening.

Installation procedure

166. Since each Gloetzl cell measures the pressures uniaxially in one position, an array of cells is used to give representative measurements around the tunnel cross section. Thus, a typical Gloetzl measurement station may consist of 5 to 10 cells located against the rock and parallel to the shotcrete surface, with an identical number of cells located perpendicular to the rock and shotcrete surfaces. The former cells monitor radial pressures (P_r) perpendicular to the shotcrete, and the latter measure tangential pressures (P_t) in the shotcrete parallel to the shotcrete surface.

167. The installation of a typical Gloetzl measurement station of from 10 to 20 cells may be accomplished in 2 to 4 hr with proper organization and sufficient labor. The Gloetzl cell assembly is assembled prior to installation and comprises individual pressure lines to each cell with a common return line. Stainless steel tubing, although more expensive than nylon, is more convenient for cramped installation underground since it bends easily and retains its shape, whereas the flexible nylon tubing tends to be awkward to handle. For handling purposes, the cell assembly may be coiled into approximately a 5-ft-diam coil.

168. When the instrument station location is chosen in the field, an exposed round may be shotcreted except for a channel 2 to 3 ft wide where the cells and lines are to be placed. Supporting pins on 4- to 6-ft centers are installed, and the Gloetzl arrangement is uncoiled, positioned, and fastened to the pins.

169. The radial pressure cells are fastened to the ground, with a pad of plastic grout between them to provide a proper bearing surface. The grout pad consists of a sand cement mix with just enough water

added to form a plastic claylike material.

170. The tangential pressure cells are installed by fastening them to short bars or anchors aligned in the direction of desired cell placement and spaced so that the cell may be placed between them with its ends resting against them. Once positioned, the cells are firmly fastened to the bars with tie wire. If the bars are made long enough to protrude from the final shotcrete surface, they provide a convenient method for locating the cells.

171. Following the positioning and fastening of the cells, the intervening hydraulic lines are firmly fastened on approximately 1-ft spacing, and the junction box is attached to the wall, again using bars and tie wire. Shotcrete is then placed, taking care to cover the cells with the shotcrete nozzle at right angles to their surfaces, to ensure a proper compaction and adhesion to the cell surfaces.

172. When the shotcrete lining has been completed but has not yet reached appreciable strength, the cells should be read to establish a zero reading. The zero reading is a function of the pressure placed in the cell during fabrication and the difference in elevation of the cell above the pump. Initially, readings should be taken before and after every blast to establish the shape of the pressure-time curve. Once the pressures begin to stabilize, readings can be taken at greater intervals.

Data reduction and presentation

173. The readings are taken and recorded, and the zero reading is deducted to give the increase in pressure. This increase is plotted versus time for each cell, and the slope of the curve produced is examined. If the slope increases over a period of several readings, an unstable condition likely exists, which could lead to local failure.

174. The measurements are generally presented as pressure distributions around the tunnel section for a given time. Both radial and tangential pressures can be shown on the same section, or alternatively, distributions of either property can be shown at different time periods.

Pressure cell measurements

175. Gloetzl pressure cell data have been reported for the following projects: CNR Vancouver Tunnel, Canada; Mexico City Tunnel, Mexico;

Schwaikheim, Regensburg, Frankfurt, and Nurnberg Tunnels in Germany; Washington, D. C., Test Tunnel, U. S. A.; Tarbela Tunnels, Pakistan; and Wolfsburg and Tauern Tunnels in Austria. The pressure cell data of these projects are presented in Table 3.

176. Of the above tunnels the Tauern Tunnel provides the most interesting example. The Tauern Tunnel is a highway tunnel approximately 11 m wide, 9 m high, and 6.4 km in length. The north portal, which shall be described here, was driven for 360 m through a detrital mountain slope of wet sand, crushed rock, clay, and large boulders. Excavation was carried with four drifts, using full spiling with steel channels in the arch. The face was supported by leaving a supporting belly of ground and by immediately shotcreting all exposed ground surfaces. Temporary support was provided initially by 50 cm of shotcrete reinforced with steel ribs and wire mesh, and at a later stage alluvial anchors were added. The multiple headings were driven so as to close the invert within 30 days.

177. Initially, instrumentation was limited to geodetic roof settlement measurements, which demonstrated the salutary effect of alluvial anchoring. By the use of this system, vertical roof movements were reduced from 200 to 50 mm under similar loading conditions. Full instrumentation stations were installed starting with sta 202.5. Radial and tangential pressures, P_r and P_t , convergence, and rock deformation were monitored. The height of cover at this station was 60 m.

178. For three weeks, the radial pressures in the right quarter-arch rose rapidly, until failure occurred in the form of a longitudinal shear fracture 20 m in length, with a 3-cm overlap of the two edges. Rupture occurred when radial pressures approached 7 kg/cm^2 at the failure location. Typically, the maximum tangential pressure of 58 kg/cm^2 was in the left quarter-arch or in the opposite side of the tunnel section. Vertical roof settlement was only 2 cm at the time of failure. The failure was repaired but little additional support was provided, whereupon P_r and P_t rose to a maximum of 18 kg/cm^2 and 100 kg/cm^2 , respectively, before equilibrium was reached 6 months after excavation. Apparently, the failure occurred because the roof was too thick and

Table 3
Summary of Pressure Cell Measurements

Name	Description	Height Cover m	Span m	Shotcrete cm	Rock- bolts	P _r Max kg/cm ²	P _r Avg kg/cm ²	P _t Max kg/cm ²	P _t Avg kg/cm ²	Comments
CNR Tunnel										
Sta 67+87	Cemented conglomerate	90	6.0	15	No	2.8	1.4	20.0	8.5	
Sta 60+21	Soft, wet, sandstone	80	6.0	15	Yes	7.7	4.8	27.0	9.6	
Sta 48+05	Brittle shale	45	6.0	15	Yes	7.4	4.8	97.0	34.0	
Schwaikheim										
Section I	Plastic clay marl	20	10.0	20	Yes	2.6	1.0	5.0	2.5	
Section II	Plastic clay marl	20	10.0	20	Yes	1.8	1.0	2.8	1.5	
Regensburg										
Sta I	Plastic clay marl	60	6.0	20	NA*	2.5	1.5	60.0	30.0	
Sta II	Plastic clay marl	60	6.0	20	NA	7.0	4.0	90.0	30.0	II weaker ground
Washington, D. C. Test Tunnel	Plastic clay	15	2.5	10	No	0.6	0.4	17.0	10.0	
Mexico City	Sandy tuff, sand	50	14.0	20	Yes	8.0	5.6	65.0	32.0	
Frankfurt Test Tunnel	Clay, sand	10	2.5	15	No	0.7	0.5	2.6	1.0	
North Tunnel	Clay, sand	10	6.7	15	Yes	1.9	1.0	18.0	14.0	Twin tunnels with 5.3-m pillar
South Tunnel	Clay, sand	10	6.7	20	Yes	2.7	1.5	25.0	15.0	
U-Bahn Nurnberg	Sandstone	7	10.0	15	Yes	--	--	30.0	20.0	
Wolfsburg	Jointed schist	NA	10.0	15	Yes	7.0	3.0	20.0	10.0	Caved ground
Tarbela Tunnels										
Sta 3	Schist, phyllite, limestone	150	22.0	20	Yes	8.0	3.0	90.0	22.0	4 transition tunnels
Tauern Tunnel										
Sta 202.5	Clay, crushed rock, boulders	60	11.0	25 to 50	Yes	18.0	8.5	100.0	70.0	
Sta 255	Clay, crushed rock, boulders	50	11.0	25 to 50	Yes	18.0	5.0	100.0	60.0	
Sta 405	Crushed phyllite	140	11.0	10	Yes	2.5	1.5	22.0	10.0	
Sta 1739	Crushed phyllite	850	11.0	15	Yes	10.0	4.0	33.0	22.0	
Sta 2195	Crushed phyllite	900	11.0	15	Yes	8.0	4.0	25.0	15.0	

* Not applicable.

rigid, and insufficient time was allowed for plastic deformation to occur before the invert was closed. These conditions were corrected, and no further failures occurred in the detrital slope section of the tunnel.

179. Sta 255, installed in similar ground conditions and with 50 m of cover, experienced very similar pressures with virtually identical distribution around the section. Equilibrium was attained after 3 months without experiencing failure. As illustrated in Figure 21, a comparison of the pressure distributions of these two stations approximately 53 m apart shows remarkable similarity.

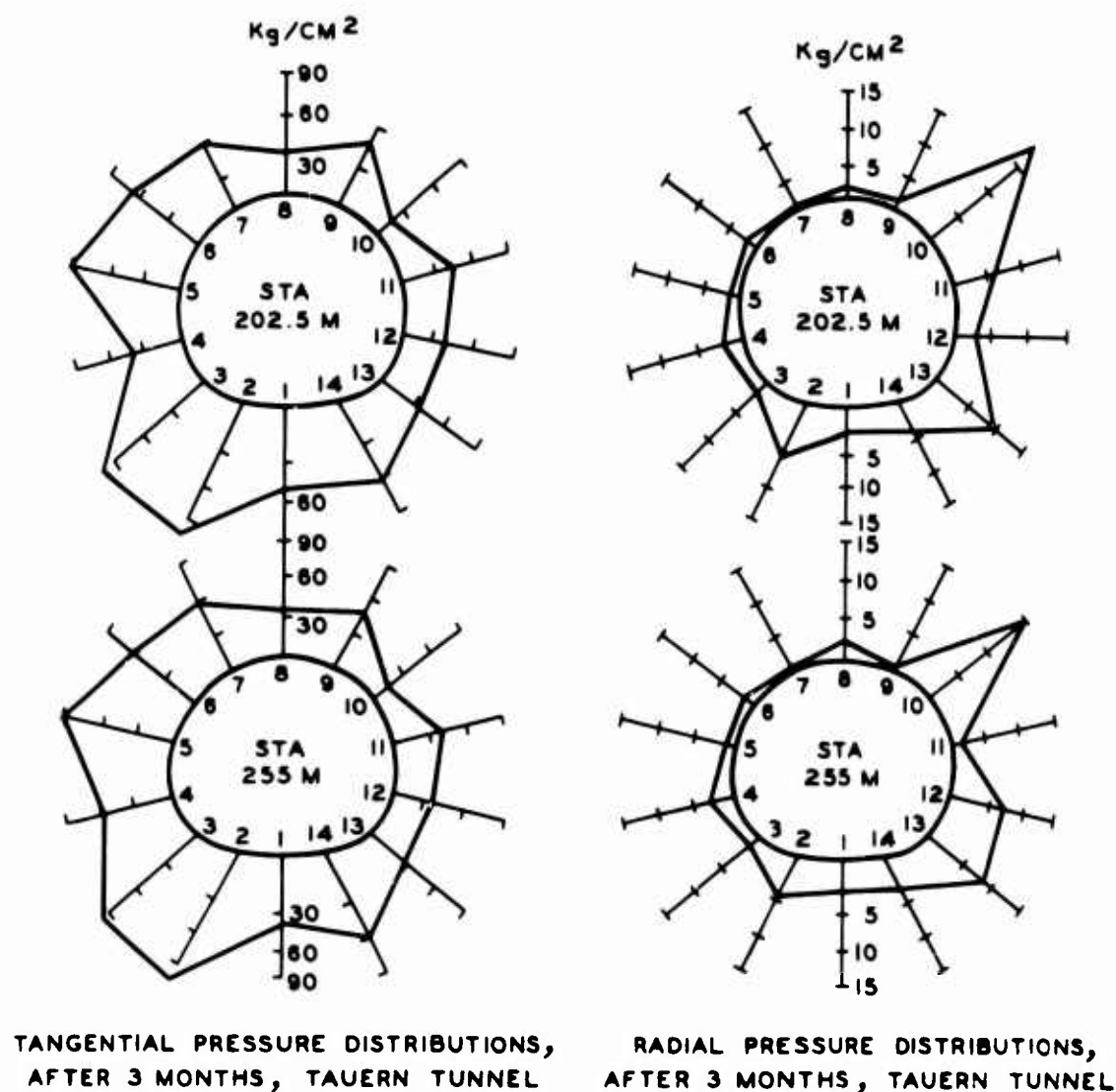


Figure 21. Comparison of radial and tangential pressure distribution for sta 202.5 m and 255 m, Tauern Tunnel

180. After the detrital slope was passed, the tunnel entered highly sheared and fractured black and green phyllites, which broke into soapy flakes upon exposure. High tectonic stresses were encountered in this formation. Advance was carried out by drilling and blasting a top heading and bench. Primary lining consisted of 10 cm of shotcrete reinforced with wire mesh and steel ribs and 4-m-long SN bolts. As the height of cover increased, the rock reacted more plastically and ruptured the thin flexible lining. However, equilibrium followed thereafter without further support, which indicated, firstly, the effectiveness of even a fractured reinforced shotcrete lining and, secondly, the far greater importance of the bolts and "rock arch" in supporting the tunnel. A solution was developed to avoid damage to the shotcrete lining by leaving longitudinal stress-relief slots in the lining to permit controlled plastic deformation to occur. These 15-cm-wide slots on occasion closed up completely after only 2 to 3 days. After 6 to 8 weeks, damage to the lining was repaired and any voids were grouted.

181. Full instrumentation stations were installed to monitor pressures, deformation, and convergence. Sta 405, with a cover of 140 m, registered radial pressures of from 1 to 2.2 kg/cm² and tangential pressures of 2.5 to 24 kg/cm². Both radial and tangential pressures showed cyclical loading due to the stress-relief fracturing that occurred in the lining. Equilibrium was reached shortly after the invert was excavated or three months after initial work in the arch. Extensometers recorded deformations of 10 to 80 mm, and up to 110-mm convergence was recorded.

182. Sta 709 was located in identical ground but with a height of cover of 350 m. Convergence measurements showed a convergence of twice as great as that at sta 405, apparently a direct result of the increased depth. Pressure data for sta 709 are not given in the reference.

183. Sta 1739 was excavated in four stages through graphitic phyllite with 850 m of cover. Primary support was 15-cm shotcrete with wire-mesh steel ribs and 4-m-long SN anchors. Radial pressures varied from 1 to 10 kg/cm², and tangential pressures varied from 1 to 33 kg/cm². Convergence measurements showed a maximum of 280 mm, and extensometers

displaced a maximum of 70 mm. Equilibrium was not attained for approximately 3 months. Sta 2195 was excavated through similar conditions as sta 1739 and similar pressures were monitored. Maximum convergence, however, only reached 120 mm.

Analysis of pressure cell measurements

184. From the available data, it may be possible to arbitrarily classify and describe the tunnels as follows:

- a. Shallow, soft-ground tunnels with less than 20 m of cover had the average P_r not exceeding 1.5 kg/cm² regardless of ground conditions, lack of anchoring, or large spans. Average tangential pressures did not exceed 15 kg/cm². Of these tunnels, the Schwaikheim and Frankfurt Test Gallery registered tangential pressures of the same order of magnitude as radial pressures, a condition not encountered elsewhere.
- b. Moderate cover tunnels with loading mainly from loosening pressure were likely exemplified by all three stations of the CNR Vancouver Tunnel. Both radial and tangential pressures were higher than one would expect, particularly for sta 60+21 and 48+05. A possible explanation for this condition may be that the 15- to 20-cm shotcrete lining, acting as both temporary and permanent support, was fairly rigid in comparison with the larger tunnels reported and that the high pressures were accompanied by correspondingly low deformations. Shattering of the rock by blasting also likely contributed to high stress values.
- c. Highly stressed, plastically behaving tunnels would include Regensberg, Tarbela, Tauern, and likely the Mexico and Wolfsburg Tunnels. Both radial and tangential pressures were high, and equilibrium was not rapidly attained. Evidence of time-dependent deformation was experienced in each of these cases through gross movements or lining ruptures.

185. The relationship between P_r and P_t is inconsistent in several instances but two patterns emerge from the assembled data: firstly, that P_t is generally an order of magnitude greater than P_r ; secondly, that their maximum values commonly occur on opposing sides of the arch. A further property of P_r and P_t is their sensitivity to work conditions several tunnel diameters away from the measurement

station, as evidenced in the CNR Vancouver and Mexico Tunnels. The great similarity between P_r and P_t for two stations 50 m apart at the Tauern Tunnel supports the reproducibility and representativity of this data.

186. At the CNR Vancouver Tunnel the variation of P_t across the shotcrete thickness was measured by installing two cells at each location in the arch. The variation was considerable, which indicates that P_t values should not be taken as totally representative of the lining pressures. Thus, the Gloetzl measurements are useful representative data, but caution should be exercised when inserting them into some design or stability analysis.

Alternate Systems to the Gloetzl Cell

187. Several alternative methods of measuring tangential or lining stresses and strains have been used in North America. These include electric embedment strainmeters and photoelastic stressmeters. The strainmeters and stressmeters can be used with thinner shotcrete linings than the Gloetzl cell, which requires a 5-in. minimum thickness. The embedment strainmeters are remote readout devices like the Gloetzl, while the stressmeters require access to the installation. An advantage of the stressmeter over the other two devices is that it is biaxial, and so measures plane stresses rather than uniaxial strains or pressures. The difficulty in relating strain to stress, as well as the problems of correction for temperature effects, would seem to discourage a more general use of the embedment strainmeters.

188. Terra Technology Corporation and Slope Indicator Company (both located in Seattle, Washington) produce a pressure cell very similar to the Gloetzl cell in operation, except that compressed air or gas is used to equalize the pressure in the cell instead of hydraulic fluid. These devices could be used in installations to measure radial pressures in a manner identical to the Gloetzl cell.

Embedment strainmeters

189. Two types of strainmeters have been used for shotcrete linings in Washington Metro construction:

- a. Vibrating wire strainmeters. The vibrating wire strainmeter consists of a pretensioned steel wire supported between two flanges and enclosed in a protective tube. An electromagnetic coil is mounted between the supporting flanges adjacent to the wire, which when energized causes the wire to vibrate at a frequency dependent on the tension in the wire. The vibrating wire in turn induces an AC voltage in the coil with the same frequency. The coil is connected by cable to a readout device, which produces the energizing current and measures the frequency of the resulting induced current. As the supporting flanges are extended or contracted due to the strains in the host material, the resulting change in frequency of the steel wire can be measured and strain computed. The vibrating wire strainmeter has good long-term stability, high accuracy, and relative insensitivity to moisture. Readings of the device are made using a frequency counter. Geonor, Telemac, and Terrametrics offer vibrating wire strainmeters.
- b. Bonded resistance strainmeters. The bonded resistance strainmeter, fabricated by the University of Illinois for use in shotcrete linings, consists of a thin steel rod enlarged at both ends with SR 4 electrical resistance strain gages bonded to its surface. Waterproofing protection is provided by plastic and neoprene rubber coverings. Measurements are taken with a digital strain indicator.

190. The embedment strainmeters are fastened to the rock surface and embedded in the initial shotcrete layer, taking care to cover them thoroughly and prevent damage to the lead wires. Lead wires are enclosed in plastic pipe and brought to a convenient location where readings may be taken.

191. The strainmeters are located in such positions on the rock surface where future shotcrete stresses are likely to occur. Detailed mapping and photos should be made of the instrument stations to aid in the correlation of strainmeter data and observed or measured displacements. Extensometers may be installed nearby to monitor block movement which would result in strains in the shotcrete liner.

192. Although the strainmeters are partially temperature compensated, temperature probes should be installed adjacent to the strainmeters to allow corrections for temperature to be made. The uncertainty of temperature compensation and other corrections discourages a strictly

quantitative use of the strainmeter data. However, the instruments do provide excellent qualitative data, in some cases difficult to obtain by other means.

193. The vibrating wire strainmeters were found to be more reliable than the bonded resistance devices as they are not so moisture-sensitive. They have the further advantage that extending the wire leads does not affect the data.

Photoelastic stressmeters

194. The photoelastic stressmeter consists of a thick-walled glass cylinder with a parabolic reflector on one end. When the stressmeter is glued with epoxy into a hole of slightly larger diameter than the cylinder and stresses are generated in the host material, the stresses normal to the axis of the cylinder are transferred to the glass plug. When viewed with polarized light, a biaxial fringe pattern forms around the central hole, and the magnitude, direction, and signs of the principal major and minor stresses can be deduced from this fringe pattern.

195. Photoelastic stressmeters manufactured by Horstman Ltd. were used to monitor shotcrete lining stresses in the Balboa Tunnel, part of the Los Angeles Metropolitan Water District freshwater supply project. The Balboa Tunnel is 16 ft in diameter, driven through mudstone, siltstone, poorly cemented sand, and conglomerate. Ground cover is 50 to 60 ft with a housing development directly over the tunnel alignment. The tunnel was excavated by a TBM with 3 in. of shotcrete being applied directly behind the cutterhead for temporary support.

196. The stressmeters were installed in the center of the arch and quarter-arch positions at each instrument station. A total of nine stations were installed throughout the length of the tunnel. The observations showed an increasing load for 10 to 14 days after installation, after which equilibrium was attained. Both tensile and compressive stresses were measured, although the former were uncommon. Compressive stresses averaged 28 kg/cm^2 , with 42 kg/cm^2 the maximum observed value. Tensile stresses of up to 25 kg/cm^2 were also observed. In two of the stations, stresses were lower than 8.5 kg/cm^2 , this being the approximate lower limit of accuracy of the installation. The variation of

stress from point to point in the arch was not generally as extreme as normally measured, possibly due to the regular surface cut by the tunneling machine.

197. Three methods of installing the stressmeter were utilized at the Balboa Tunnel. The manufacturer-recommended procedure is to drill a smooth hole with a diamond coring bit to a diameter $1/8$ in. larger than the meter and to cement the stressmeter into the hole with epoxy cement. The second method of installation was to form a pad of shotcrete mix around the stressmeter and, after the pad had hardened, to spray it into the shotcrete lining. A third method, consisting of holding the stressmeter and spraying it into the lining, was not successful. Perhaps the best solution would be to epoxy the stressmeter into a formed pad and then to spray the pad into the lining. The latter method allows the precise work of cementing the gage into the pad to be carried out under controlled conditions, with the epoxy providing a more satisfactory bond than the shotcrete.

Terra Technology and Slope Indicator Total Pressure Cells

198. In order to measure radial pressures a Total Pressure Cell system can be used as an alternative to the previously discussed Gloetzl system. The Total Pressure Cell apparatus consists of a circular sensing pad, connecting tubing, and a pneumatic sensing device. The cell measures radial pressures when attached directly to the rock surface with epoxy and buried with shotcrete. This installation is very similar to the Gloetzl radial pressure cells. Tubing for the cell is heavy-wall jacketed PVC, which may be attached to the wall and buried with shotcrete. The tubing is brought to a junction box at a convenient location and protected with a metal cover. Although presently not manufactured, there seems little reason why rectangular cells, which would be convenient for measuring tangential pressures, could not be fabricated. Their installation would be identical to that of a Gloetzl cell.

Borehole Extensometers

199. Borehole extensometers are devices which measure axial

displacement between fixed anchors and the instrument head by means of rigid rods or tensioned steel wires. The displacement is monitored either with a precision feeler gage or by some electrical, remote readout device. Multiple-position borehole extensometers (MPBX) have several anchors and individual parallel wires or rods leading to individual sensing elements in the extensometer head.

200. Extensometer readings are measurements of anchor movement relative to the extensometer head, which is also moving. For data presentation, displacements are transposed to present movement of the extensometer elements relative to the deepest anchor. If installed at sufficient depth, typically 2 or 3 tunnel diameters away from the tunnel, the bottom anchor can be assumed to be fixed, and the displacement relative to it to be absolute. However, often practical considerations prohibit the use of such long boreholes, and therefore the measured displacements are relative or approximate values.

201. The deepest anchor in an MPBX installation is used as a reference point, and shallower anchors are distributed between the deepest anchor and the collar, with a concentration nearest the collar where the greatest displacement occurs. The precise location and number of anchors required depend on geological conditions such as fracture density, adjacent known discontinuities such as faults or contacts, support requirements, and construction restraints. Generally, four to six anchors are sufficient to monitor the most complex conditions. Redundant anchors provide a useful check on each other, however. For example, if one anchor acts in an aberrant manner compared to the adjacent anchors, its behavior is likely due to some malfunction and the data may be disregarded.

202. The extensometer head must be well protected against damage from flyrock and other construction hazards. A recess can be excavated in the rock for the instrument head to be installed, and a steel cover plate bolted to the rock. If subsequent shotcreting takes place, a wooden form may be placed around the cover plate to prevent its being buried. Surface installations may be conveniently made inside a concrete manhole.

203. Wire-type extensometers are subject to the effects of wire sag, friction on the wire, elongation of the wire, changes in wire modulus, and temperature effects. By proper design and installation, these effects can be minimized. Friction on the wire and wire sag are reduced to a minimum if the wires are tensioned to 10 lb or greater, if they are installed straight in the hole without touching other wires, and if their contact points are bushed with a material such as nylon or teflon.

204. Elongation of the anchor wires cannot be eliminated, but this elongation can at least be made approximately constant if the wires are tensioned by a constant-tension spring. When the wire is coiled at the fabricator, the Young's modulus of the anchor wire is altered and must be passed through a wire straightener to produce a uniform modulus. Such straightening also decreases wire sag and reduces the chance of entanglement. During installation, the wires must be held taut and in position by a suitable jig to avoid entanglement in the boreholes.

205. Temperature effects due to the differences in coefficients of expansion of the rock and the anchor wire are minimal in underground application. However, extensometers mounted at surface and subject to seasonal changes in temperature must have corrections applied to the data.

206. Rod-type extensometers are preferable to wire-type devices, because the rods are not tensioned and friction effects can be readily avoided by encasing the rods in individual polyethylene tubes. The principal drawback of rod extensometers is inconvenience of installation in cramped working conditions underground. Furthermore, with the exception of the devices described below from the suppliers, Stitz (Buchholz, West Germany) and Interfels (Salzburg, Austria), no remote readout rod extensometers have been developed.

207. Remote readout wire extensometers are generally used when subsequent construction causes access to the instrument to become difficult or impossible. In the case of large underground cavities such as power plants or subway stations, extensometers are frequently installed from pilot or top headings to monitor movements throughout the

excavation of the complete section. In such cases, either remote readout devices are required with suitably protected readout cables extended to the excavation floor, or access must be provided to the instrument head by means of scaffolding or by hydraulic crane. The latter solution requires careful writing of contract specifications and the complete cooperation of the contractor in order to take readings at the appropriate times.

208. The reliability of electrical MPBX has been occasionally unsatisfactory. Improved waterproofing of the instrument head and the use of stricter specifications for electrical cables and connectors would partially alleviate this problem.

209. Various types of anchor systems are offered by manufacturers. For an anchor to be effective, it must provide complete permanent anchorage, it must not be capable of skewing in the hole to block passing anchor wires, and it should provide nylon- or teflon-sleeved openings for deeper anchor wires to pass through. Individual sleeves for each rod or wire would be preferable to a central hole to eliminate friction between the individual wires.

210. Grouted anchor systems are often used, particularly when the borehole is drilled in soft or unstable ground which would block the hole if it were not grouted. Although grouted systems provide excellent long-term protection to the borehole installation, there are several reasons why they are not used more extensively: inconvenience of providing grouting equipment, time delays due to grouting, and difficulties of completely filling up holes. Furthermore, grouted anchor systems are somewhat insensitive to low-level compression strains.

211. To summarize, each extensometer type will serve an individual function. For normal underground work, the double-position rod extensometer used singly or in groups provides low-cost reliable data with a minimum of interruption to the construction sequence. If access to the measurement head is not available or if other reasons dictate the use of remote readout devices, the electric wire-type MPBX must be used. For installations from the ground surface, a rod-type MPBX provides the most reliable and convenient solution. Grouted anchor systems are

recommended for holes through overburden or other loose material that may block. For other conditions, a positively acting, hydraulically actuated anchor is recommended.

Instrumentation of
Washington Metro Projects

212. Double track tunnel. The Rock Creek Park to Dupont Circle Tunnel is 30 ft wide, 22 ft high, and 5000 ft long and passes through jointed schistose gneiss. Temporary support comprised of an initial 2-in. layer of shotcrete was followed by the installation of 8- and 12-ft-long rockbolts or steel ribs if ground conditions warranted. Final or permanent support was installed several hundred feet behind the heading and consisted of additional layers of shotcrete reinforced with wire mesh. In steel rib-supported sections of tunnel, the ribs and timber blocking were buried in the final lining, resulting in shotcrete thicknesses of from 1.5 to 5 ft depending on overbreak.

213. A very extensive instrumentation program was established with the purpose of monitoring conditions during construction and providing information which could be utilized for design of further Metro Tunnels. A third function of the instrumentation program was to evaluate the utility of and mode of support provided by a relatively thin shotcrete lining under the prevailing geological conditions. The instrumentation program was accompanied by a detailed observational program which defined and recorded the geologic conditions, primarily jointing, which contributed to support loads and construction difficulties.

214. Instrumentation in the tunnel consisted of extensometers and embedded strainmeters. In addition, multiple-position extensometers were installed from the ground surface prior to advancing the tunnel. These instruments were installed in inclined holes which intersect the tunnel in the quarter-arch positions. Anchors were installed at distances of 1, 2, 4, 8, 16, and 32 ft from the future tunnel arch and grouted in place.

215. Two test sections were installed in the running tunnel. At Test Section I, 26 strainmeters were embedded in the initial shotcrete lining and 11 double-position extensometers installed in the tunnel arch.

Extensometer anchors were placed from 3 to 24 ft from the excavated rock surface. The embedded strainmeters were either Telemac vibrating wire strainmeters or bonded resistance strainmeters fabricated by the University of Illinois. The double-position extensometers were rod-type extensometers with standard expansion anchors, also fabricated by the University of Illinois.

216. In addition to the two measurement stations, double-position extensometers were installed throughout the length of the tunnel to monitor displacements under different geologic and support conditions. The behavior of the rock arch under combinations of shotcrete, rockbolt, and steel rib support was investigated in this manner.

217. Instrumentation results were closely related to local geologic conditions such as joints, shear zones, and block size. Embedded strainmeters verified strain conditions associated with the movement of rock blocks visible in outline. Extensometers installed through these blocks further correlated this observational and measured data. Strains in the order of 50 to 150 microstrains were measured both in compression and in tension. Near a visible crack, tensile strains of 150 to 300 microstrains were recorded. Rock displacement measured by the extensometers was 0.02 to 0.03 in. in the rockbolt- and shotcrete-supported sections of tunnel and greater than 0.10 in. in the steel rib-supported tunnel.

218. Dupont Circle Metro Station. The Dupont Circle Station is a cavity 775 ft long, 77 ft wide, and 44 ft high excavated in schistose gneiss. Rock cover is approximately 25 ft with a further 35 ft of overburden. The rock is moderately foliated and contains several well-developed steeply dipping joint planes. The cavern design provided a temporary-permanent support system comprised of shotcrete, rockbolts, and steel ribs. A central drift was driven initially, supported by shotcrete and 24-ft anchors in the crown. Anchors were also inclined out from the central drift to reinforce the roof above the headings to be driven on either side. Two wall-plate drifts were then excavated, from which further bolting was accomplished. The remaining two top heading drifts were then removed and supported with shotcrete, bolts,

and steel ribs resting on the wall-plate haunches. The steel ribs were blocked to the shotcrete with further shotcrete and embedded with a final shotcrete layer. The bench was removed in three stages: initially removing the bulk of the core and subsequently trimming the two sidewalls. The sidewalls were supported with shotcrete and rockbolts.

219. Instrumentation of the station was initiated before the excavation contract was awarded. At this time, a pilot heading was driven in the center of the crown through the entire length of the station to permit inspection of the rock conditions. Slope Indicator wire-type MPBX's were installed from the pilot heading, as were rod-type double-position extensometers made by the University of Illinois. In addition to the pilot tunnel installations, rod-type MPBX's fabricated by the University were installed from ground surface to intersect the future station in the quarter-arch positions. All of these extensometers were designed to monitor displacements in the station crown throughout its entire construction.

220. The instrumentation installed during construction included four test sections, of which two contained a greater concentration of instruments than the others. The major test sections comprised an array of double-position rod extensometers in the arch and sidewalls and strainmeters attached to the steel ribs. The extensometers were grouped together in clusters to monitor displacements at different depths at the same location. Strainmeters were installed in pairs at various locations on the steel ribs to measure strain and, by calculation, stress. Temperature probes were installed near the strainmeters to allow temperature compensation to be made to the strain readings. In addition to the instruments installed during construction, the MPBX installed from ground surface intersected the excavation at these major test stations, as did several of the pilot tunnel instruments. A major instrumentation test section is shown in Figure 22.

221. Measured displacements were generally less than 0.2 in. with several important exceptions occurring in the center of the crown where mass movements of up to 0.4 in. were measured. One anchor near the borehole collar registered a maximum of 2.2-in. displacement. Sidewall

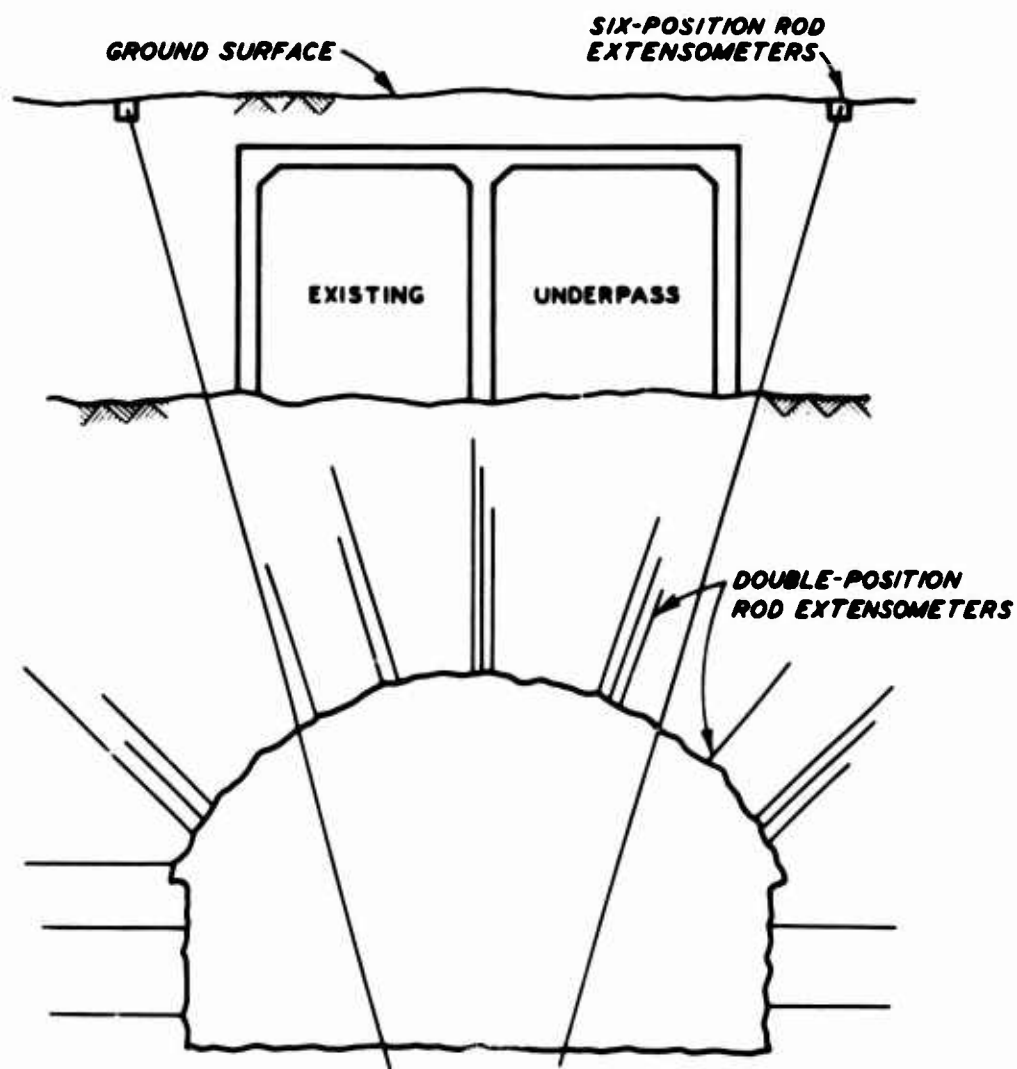


Figure 22. Typical measurement section at Dupont Circle Metro Station

extensometers generally did not exceed a 0.2-in. displacement. The extensometers provided valuable up-to-date information for use during construction.

Analysis of extensometer results

222. Extensometer data produce valuable information on the behavior of the surrounding rock as a result of construction operations. Information on the displacement-time relationship, the relative displacement of individual anchors, the magnitude of displacement, and the increasing or decreasing rate of displacement essential.

223. By plotting the individual anchor displacement versus time, the attainment of equilibrium can be observed. The time required for an opening to reach equilibrium to some extent depends on the quality of the ground, but, for example in the Tarbela Tunnel, equilibrium can take as much as 17 months to attain equilibrium. On the other hand, in the Washington Metro Tunnels, movement generally tapered off a few days after excavation. Thus, although the period required to reach equilibrium may not be of primary interest, the attainment of that stability is a valuable reassurance.

224. By observing the displacement-time graphs, it may be noted where major displacement is occurring and what importance to attach to it. While appreciable displacement confined to near-surface locations would indicate a normal loosening condition, deeper-seated displacements could indicate a fundamental support deficiency or block separation at a defined plane of weakness. By observing the depth of important displacement, for example, the design length of rockbolts may be reappraised.

225. The magnitude of displacement depends on a number of factors, including the existing stress condition, rock strength, rock mass defects, size of the opening, construction procedures, and type of support system utilized. The resulting displacements might, therefore, be elastoplastic in nature or the result of the movement of rock blocks. It was observed in the Tarbela Tunnel that a radial deformation of 4 in. was required to rupture the lining. Deformations of a shotcrete and steel rib-supported tunnel at Granduc Mines reached 9 in. over a 2-yr period without noticeable cracking of the shotcrete. On the other hand, it was noted in the Washington Metro Tunnels that a thin shotcrete lining ruptured where displacements resulting from block movement exceeded 0.05 in.

226. A further factor to be considered is that the displacement measured by the extensometer is only that component of total movement in a direction axial to the borehole. Therefore, a small anchor displacement may be indicative of much larger movements occurring on planes intersecting that of the borehole. Hence, it is important to consider all available geological information in analyzing extensometer data.

227. Instrumentation installed prior to excavation from surface, or during the initial stage of multiple-heading excavation will initially show a rising rate of displacement when excavation approaches the instrument. However, the rate of change of displacement should begin to taper off soon after excavation passes. An increasing rate of displacement indicated by the slope of the displacement-time curve should be carefully observed, as it may be an indication of impending structural failure. Depending on geological and site conditions, the acceleration of displacement may suggest immediate reinforcement of the tunnel support system.

Wire extensometers

228. Terrametrics MPBX Model F-2. The Terrametrics MPBX F-2 is an 8-channel electrical readout wire extensometer. The extensometer head consists of steel cantilevers to which the ends of the anchor wires are attached. The strain in each cantilever caused by anchor displacement is measured by electrical resistance strain gages bonded to the cantilever surface. By calibration, this strain may be converted to extension of the wire due to relative movement between the anchor and the instrument head. The strain gages are electrically connected to a multiple connector on the extensometer head. The instrument is monitored with a multichannel digital strain indicator connected to the extensometer head with a multistrand cable.

229. Teledyne Terrametrics (Golden, Colorado) offers four types of anchors to be used with the F-2 electric MPBX and the CSLT mechanical readout devices. The FWS anchor consists of a flat spring-loaded wedge which expands against the surface of the borehole when the anchor wire is tensioned, or alternately when a base-screw is tightened. The GW anchor is a simple grooved anchor which is grouted in place. The SLW anchor has three spring-loaded claws which grip the surface of the borehole when the actuating trigger is pulled. The HG anchor is an aluminum cylindrical shell wound about with flattened copper tubing. When pressured with a hydraulic pump, the copper tubing inflates and expands against the wall of the borehole.

230. Slope Indicator rectilinear extensometer. The extensometer

head of this device consists of six single-turn potentiometer sensing elements encased in a protective box. The wires from the individual anchors are tensioned by a constant-force spring and are then passed over a pulley attached to the potentiometer shaft. The single turn of the potentiometer shaft amounts to approximately 2 in. of anchor wire travel. The potentiometers are attached to electrical leads combined at a multiple socket to which a multistrand cable is attached. The instrument is monitored by means of a multichannel digital strain indicator.

231. Anchors may be the single- or double-claw hydraulic types or the grouted type. The single or double hydraulically actuated anchors have three or six prongs, respectively, which are forced out of their enclosing cylinder and against the rock, deforming the prongs in the process. The double-claw anchor has less tendency than the single-claw model to skew in the hole and thereby risk entanglement with anchor wires from deeper anchors. The grouted anchors consist of pipe connections joined by flexible polyethylene tubing which encloses the anchor wires and protects them during grouting.

232. Terrametrics mechanical readout MPBX. This device is available as 6- or 8-anchor devices, models 6 CSLT or 8 CSLT, respectively. Each anchor wire is attached to a plunger mounted over a coil spring, which provides tension to the anchor wire. A flat measuring point cap is placed on the tip of each plunger, and readings are taken between these points and a machined measurement table. Readings are taken with a rod feeler dial gage. The four types of anchors previously discussed, models FWS, SLW, GW, and HG, can be used with the CSLT model extensometers.

233. Peter Smith Mark II MPBX. The Peter Smith extensometer is a wire-type device with up to six positions. The most expensive part of the device, the extensometer head and readout device, is incorporated in a portable form which may be used for any number of borehole installations. Readings are taken by gripping the end of the individual wire in the extensometer jaws, tensioning the wire to a given tension as shown on the dial gage, and reading the displacement on a micrometer scale. Each anchor wire is positioned and held taut in a slotted jig when not

being measured. The device was developed by the University of Newcastle-upon-Tyne and is marketed on this continent by Terra Science Ltd. (Vancouver, B. C., Canada) and Slope Indicator.

234. A hydraulically set wedge anchor, marketed by Terra Science, is often used with the Peter Smith extensometer. The anchor consists of two sliding wedges fitting into slots in a cylindrical anchor piston. The anchor piston fits into a cylinder-shaped setting tool and is secured with a steel shear pin. The anchor is set by positioning the anchor in the hole and pumping up the setting tool with a hydraulic pump connected through 1/2-in.-diam aluminum pipe. The hydraulic ram in the setting cylinder forces the piston forward, thus causing the two sliding wedges to grip the walls of the borehole. At a predetermined pressure, the shear pin breaks and the setting-tool cylinder may be removed. The anchor is solidly set and is prevented from relaxing by the friction of the wedges in the grooves. Each anchor has nylon-bushed guide holes for deeper anchor wires to pass through.

Rod extensometers

235. Double-position extensometers (DPBX). Double-position rod extensometers have been manufactured by Terrametrics, Slope Indicator, Soiltest (Evanston, Illinois), and the University of Illinois. Since they are relatively simple to fabricate from available rockbolt supplies, some contractors have chosen to make their own DPBX in jobsite workshops. The instrument consists of a bottom-of-the-hole anchor shell attached to a 1/4-in. rod, an intermediate anchor shell attached to a sleeve through which the first anchor assembly slides freely, a collar anchor, and a measuring head. The intermediate anchor assembly slides through the collar anchor assembly. Displacements are measured between the flat measurement surface attached to the collar anchor and the machined ends of the intermediate anchor sleeve and the bottom anchor rod. The bottom and intermediate anchors are standard expansion shell anchors such as Bethlehem K-4 or Type "C". The collar anchor is a sliding wedge type such as the Star-brand shell. Measurements are taken with a depth micrometer, which is inserted into holes drilled into the measurement head.

236. Slope Indicator MPBX. Slope Indicator Company has developed a four-position rod extensometer which may be used with either grouted or hydraulic anchors. The system comprises a central 1/4-in. rod encased in a 1/2-in. polyethylene tube with three similarly encased rods fastened about the central tube with cable ties. Both the rod and the encasing tube thread into the anchor assemblies, whether hydraulic or grouted. The space between the rod and the tubing is filled with hydraulic oil to reduce friction and prevent the tubing from collapsing when the hole is grouted. Measurements are taken with a micrometer depth gage fitted with an adapter into which the end of the anchor rod fits. The end of the adapter rests against the machined aluminum head.

237. Interfels DPBX. This device has individual anchor rods which slide within protective sleeves and are attached to grouted anchors. Segments of rod and protective sleeve are coupled together to reach lengths of up to 200 m, if required. The measurement head is fixed to a mounting plate bolted directly to the rock surface with short expansion anchor bolts. Measurements are taken with a dial gage probe or a remote readout electrical device.

238. Stitz MPBX. The Stitz extensometer can be used for up to five measurement points. The bottom anchor is connected to a solid rod, and the remaining anchors are connected to sleeves which are able to slide over each other. The anchors themselves are sliding wedge mechanical anchors which expand against the borehole walls when the anchor rod or sleeve is twisted. Rods or sleeves from deeper anchors pass through the center of the anchor-sleeve assemblies without interfering in their movement. Measurements may be made with a dial feeler gage or with an electronic readout device.

239. Soiltest MPBX. The Soiltest RM-580 has six rods and anchors. Anchors consist of sliding piston devices loaded with epoxy cement, which is extruded when the anchor is pulled backwards. The anchor is equipped with sheet metal tabs angled to catch the sides of the borehole and cause the piston assembly to collapse when the anchor is pulled backwards. Measurements are taken with a feeler rod dial indicator gage, which measures the distance from the machined end of the measurement

rods and the reference face of the sensor head.

240. University of Illinois. These extensometers are the six-position devices installed from surface to the Dupont Circle Metro Station. The longest rod is 75 ft in length. Anchors consist simply of 8-in. lengths of 3/4-in.-diam reinforcing bar, to which 3/8-in.-diam steel rods are attached. The rods are encased in 3/4-in.-diam polyethylene pipe taped to the rebar anchors to prevent the entry of grout. The measurement head consists of a base plate bolted to a concrete floor slab and a measurement surface through which the feeler gage of the micrometer readout gage passes. Six holes drilled into the measurement surface are aligned with similar holes in a guide plate, and machined steel caps are mounted on the ends of the individual anchor rods.

Field extensometer calibration

241. A field calibration test should be performed for the Terra-metrics F-2 and CSLT models and the Slope Indicator rectilinear extensometer to determine calibration factors relating to the instrument head and wire stretch, friction, and sag. The procedure entails jacking the head away from the collar anchor a measured distance and calculating calibration factors from the difference in measured anchor displacement. The test does not strictly duplicate field conditions, however, since all the wires are stretched equal amounts simultaneously. Unless the wires are separated, this simultaneous tensioning produces unusual frictional effects which may become very important for long anchor wires.

242. The Peter Smith portable extensometer should be calibrated periodically by suspending a standard weight from the measurement jaws and checking the dial gage reading. Rod extensometers, due to their direct measurement system and lack of friction effects, do not require field calibration.

Recommendations for Shotcrete Research

243. The use of shotcrete is a fairly recent innovation, particularly on this continent, and many improvements need to be made. Some of these improvements simply involve the acceptance of methods and equipment

developed elsewhere with the modifications necessary to adapt these to local conditions. On the other hand, some of the more interesting innovations have been initiated here and should be pursued. The following are some of the most urgent needs to be met:

- a. Instrumentation for shotcrete-lined tunnels should be improved to encourage its use and increase its effectiveness. The double-position extensometer is the most practical present extensometer but reliable remote-readout sensing devices should be developed for it. Alternatives or improvements to the Gloetzi cell must be developed to decrease instrument cost.
- b. New equipment, utilizing the best ideas of the Swedish contractors, should be developed for shotcreting. Compact, high-output wet-mix machines should be combined with precision remote-controlled shotcreting nozzles to allow a continuous lining process compatible with modern tunneling machines.
- c. Admixtures must be developed to produce nearly instantaneous shotcrete setting times without appreciable loss of long-term strength. Tests with regulated-set cement indicate that such results may be achieved without admixtures.
- d. The use of steel wire or fibers must be developed to increase the tensile strength and short-term ductility of shotcrete. Such properties would be useful and would allow the elimination of the use of wire mesh with shotcrete.
- e. Support systems which replace the present practice of combining conventional steel ribs with shotcrete must be developed and accepted. Austrian engineers have used lightweight steel ribs, deformed out of line and shape to conform to the rock outline, in extremely poor ground conditions. Their concept of the steel ribs acting as reinforcement rather than as conventional ribs is sound and more economical.
- f. Improved ventilation methods, perhaps similar to those employed with tunneling machines, should be developed and employed to provide an acceptable tunnel environment during shotcreting.

Rational Approach to Tunnel Support Design

244. Although presently in its infancy, the rational design of tunnel support systems appears to be developing for many of the

conditions encountered underground. Useful theories and design procedures exist for soft-ground tunnels at shallow depths, deep soft-ground or otherwise plastically acting tunnels, and hard-rock tunnels without major discontinuities. The design of supports for rock tunnels where jointing or fracturing leads to the formation of loosening pressures remains in the rule-of-thumb category, however. It seems unlikely that such design methods will be rationalized, at least until new developments such as smooth blasting and machine boring eliminate some of the variables introduced by construction procedures.

245. The present practice of designing flexible tunnel liners to utilize the passive pressure of the ground to resist thrust loading is an excellent example of current attempts to rationalize design. Considerable research is being carried out to improve the understanding of such lining-ground interaction, notably by the University of Illinois. The application of such design methods appears most applicable to design of tunnel liners in soil or soft rock at relatively shallow depth.

246. The designers of the NATM utilize a plastic design procedure extensively described in the literature. The success of the NATM is testimony to the soundness of its design basis, but further work needs to be done in this field. For example, a detailed finite element analysis of the Turbela Tunnel, executed after construction, produced disappointing results. The calculated stress values differed substantially from measurements, due to the complexity of the geology and the inability to introduce the timing of construction sequence into the calculation.

247. The design of large underground structures such as powerhouses has developed considerably over the last few years. A recent example of excellent correlation between displacements predicted from preconstruction rock mechanics tests and observed deformations is reported for the Mica Creek Powerhouse in Canada. This cavity was constructed in fairly sound metamorphic rocks under an 800-ft maximum cover. Elastic theory and finite element analysis are useful tools in the design of a cavity in such conditions.

248. The design of rock tunnels where pressures due to loosening

predominate presents a difficult problem in that small-scale geological features, different support systems, and construction methods greatly affect the loads which will ultimately bear on the lining. The presently used rules-of-thumb proposed by Terzaghi, while adequate for steel rib supports, are wholly inappropriate for rockbolt or shotcrete supports. These rules-of-thumb have been shown to be also inaccurate for steel rib support, but proposed improvements are slow to gain acceptance. The greater precision which the use of RQD values provides to describe rock conditions will likely lead to improved rules-of-thumb. However, RQD values must be carefully interpreted for formations other than igneous rocks.

249. Until some of the drastic effects of construction methods and quality of workmanship can be reduced, for example by the use of tunneling machines and smooth blasting techniques, the design of tunnel supports will have to remain on the conservative side. With new techniques being universally utilized, the application of more precise, less conservative design procedures would be possible.

REFERENCES

1. Proctor, R. V. and White, T. L., "Rock Tunneling with Steel Supports," 1946, Commercial Shearing and Stamping Company, Youngstown, Ohio.
2. Sonderegger, A., "Spritzbeton im Stollenbau," Schw. Bauzeitung, Vol 74, No. 14, 1956, pp 210-212.
3. Rabcewicz, L. v., "Die Ankerung im Tunnelbau ersetzt bisher gebräuchliche Einbaumethoden," Schw. Bauzeitung, Vol 75, No. 9, 1957, pp 123-131.
4. Mahar, J. W., Gau, F. L., and Cording, E. J., "Observations During Construction of Rock Tunnels for the Washington, D. C., Subway," Proceedings, First Rock Engineering Tunneling Conference, 1972, pp 659-681.
5. Linder, R., "Technologie des Spritzbetons," Beton u Stahlbetonbau, Vol 58, No. 2, 1963, pp 40-45 and 63-67.
6. Deere, D. U. et al., Design of Tunnel Liners and Support Systems, 1969, University of Illinois, Urbana, Ill.
7. Peck, R. B., "Deep Excavations and Tunneling in Soft Ground," Proceedings, Seventh International Conference on Soil Mechanics and Foundations Engineering, Mexico (State of the Art Volume), 1969, pp 225-281.
8. Lombardi, G., "Zur Bemessung der Tunnel-Auskleidung mit Berücksichtigung des Bauvorganges," Schw. Bauzeitung, Vol 89, No. 32, 1971, pp 793-801.
9. _____, "Tunnel Support," Proceedings, Third International Congress on Rock Mechanics, Denver, Colo., Vol IB, 1974, pp 1518-1528.
10. Obert, L. and Duvall, W. I., Rock Mechanics and the Design of Structures in Rock, Wiley, New York, 1967.
11. Kastner, H., Statik des Tunnel und Stollenbaues, Springer, New York, 1971.
12. Louis, C., "Construction de Tunnels par la nouvelle Methode Autrichienne," Revue de l'Industrie Minérale, Cahier 4, 1972, pp 22-38.
13. Rabcewicz, L. v., "The New Austrian Tunneling Method," Water Power, 1964, pp 453-457.
14. Rabcewicz, L. v. and Golser, J., "Principles of Dimensioning the Supporting System for the New Austrian Tunneling Method," Water Power, Vol 25, No. 3, Mar 1973, pp 88-93.
15. Rabcewicz, L. v., "Diskussionsbeitrag zu den Vorträgen von H.

- Becker und F. Pacher," International Symposium on Underground Openings, Lucerne, 1972, pp 561-567.
16. Linder, R., "Spritzbeton im Felshohlraumbau," Bautechnik, Vol 40, No. 10, 1963, pp 326-331.
 17. Muller, C., "New Swiss Tunnel Lining System," Reprint, NZZ, No. 282, 1971.
 18. Alberts, C. and Backstrom, S., "Instant Shotcrete Support in Rock Tunnels," Tunnels and Tunneling, Vol 3, No. 1, 1971, pp 29-32.
 19. "Use of Shotcrete for Underground Structural Support," Proceedings, Engineering Foundation Conference, Berwick Academy, South Berwick, Maine, July 16-20, 1973, American Concrete Institute Publication SP-45.
 20. American Concrete Institute, "ACI Standard Recommended Practice for Shotcrete," ACI 506-66, Detroit, Mich.
 21. Mahar, J. W., Parker, H. W., and Wuellner, W. W., "Shotcrete Practice in Underground Construction," DOT Report FRA-OR and D 75-90.
 22. U. S. Bureau of Reclamation, "Construction and Engineering Data, Water Conveyance Tunnels, Gravity and Pressure," Jan 1961, Washington, D. C.
 23. Bureau of Reclamation Engineering and Research Center, Office of Design and Construction, "Tunnels - Machine Excavation - Rate of Progress - Machine Data," Report No. REC-ERC-74-7, May 1974, Denver, Colo.
 24. Tunnels and Tunneling, London, England.
 25. U. S. National Committee on Tunneling Technology, "Tunneling Technology Newsletter," Washington, D. C.
 26. Sattler, K., "New Austrian Tunnel Construction Method, Part II, Static Forces and Design," Der Bauingenieur, USBR Translation No. 781, Vol 40, No. 8, 1965, pp 289-301.
 27. Mason, R. E., Instrumentation of the Shotcrete Lining in the Canadian National Railways Tunnel, M. S. Dissertation, May 1968, University of British Columbia, Vancouver, B. C.
 28. Leere, D. U. et al., "Design of Surface and Near-Surface Construction in Rock," Proceedings, Eighth Symposium on Rock Mechanics, The American Institute of Mining, Metallurgy, and Petroleum Engineers, Inc., New York, 1967, pp 237-302.
 29. Austin, W. G. and Fabry, J. W., "Rock Behavior Studies During Drill-Blast and Machine-Bored Tunneling," paper presented at the Third International Congress on Rock Mechanics, Denver, Colo., Sep 1-7, 1974.

BIBLIOGRAPHY

Atrott, G., "The Application of the New Austrian Tunnel Construction Method to the Construction of the Underground Railway in Frankfurt/Main," Feb 1972, Baumaschine und Bautechnik (Translated by Meynadier Co., Ltd).

Bawa, K. S., "Design and Instrumentation of an Underground Station for Washington Metro System," International Symposium on Large Underground Openings, Oslo, 1969.

_____, "Development of Shotcrete for Metro Construction in Washington," Use of Shotcrete for Underground Structural Support, Special Publication SP-45, pp 33-49, 1974, American Concrete Institute, Detroit, Mich.

Blanck, J. A., "Balboa Outlet Tunnel," 1969, Monolithic Rock Systems, Ltd.

Brekke, T. L., "Shotcrete in Hard-Rock Tunneling," Bulletin, Association of Engineering Geologists, Vol IX, No. 3, 1972, pp 241-264.

Brekke, T. L. and Selmer-Olsen, R., "Stability Problems in Underground Construction Caused by Montmorillonite-Carrying Joints and Faults," Engineering Geology, Vol 1, No. 1, 1965, pp 3-19.

Cording, E. J., "Geological Considerations in Shotcrete Design," Use of Shotcrete for Underground Structural Support, Special Publication SP-45, pp 175-199, 1974, American Concrete Institute, Detroit, Mich.

Fox, K. H., "Use of Shotcrete from the Standpoint of the Contractor," Use of Shotcrete for Underground Structural Support, Special Publication SP-45, pp 22-28, 1974, American Concrete Institute, Detroit, Mich.

Golser, J., "Practical Examples of Empirical Dimensioning of Tunnels," Rock Mechanics Supplement No. 2, 1973, pp 225-241.

Hawkes, I., "Photoelastic Stressmeter," 1967, Terrametrics, Inc., Wheatridge, Colo.

Jones, R. A. and Mahar, J. W., "Instrumentation to Monitor Behavior of Shotcrete Support Systems," Use of Shotcrete for Underground Structural Support, Special Publication SP-45, 1974, American Concrete Institute, Detroit, Mich.

Lauga, H. and Weir-Jones, I., "The Use of In-Situ Moduli Determinations as an Aid to Underground Opening Design and Their Relationship to Laboratory Determined Values," Dec 1973, Montreal, Canada.

Mason, E. E. and Mason, R. E., "Shotcrete Support with Special Reference to Mexico City Drainage Tunnels," Rock Mechanics, Vol 4, 1972, pp 115-128.

McClure, C. R., "Use of Shotcrete from the Standpoint of the Designer," Use of Shotcrete for Underground Structural Support, Special Publication SP-45, pp 18-21, 1974, American Concrete Institute, Detroit, Mich.

Miner, G. M., "Use of Shotcrete from the Standpoint of the Owner," Use of Shotcrete for Underground Structural Support, Special Publication SP-45, pp 13-17, 1974, American Concrete Institute, Detroit, Mich.

Muller, G., "Geomechanical Measurements by Interfels in the Tauern Highway Tunnel," Interfels Messtechnik Information, 1973.

"The New Austrian Tunneling Method - Der Monierbauer," Publication of Beton und Monierbau A. G., 1973.

Parker, H. W., "Current Field Research Program on Shotcrete," Use of Shotcrete for Underground Structural Support, Special Publication SP-45, pp 330-350, 1974, American Concrete Institute, Detroit, Mich.

Paul, S. L. et al., "Research to Improve Tunnel Support Systems," D.O.T. Report FRA-ORD & D74-51, Jun 1974.

Rabcewicz, L. v., "Dimensioning of Underground Excavations," Rock Mechanics and Engineering Geology, Vol 113-114, 1963.

_____, "The New Austrian Tunneling Method," Water Power, Nov-Dec 1964, Jan 1965.

_____, "Stability of Tunnels Under Rock Load," Water Power, Jun-Jul-Aug 1969.

Rabcewicz, L. v. and Golser, J., "Application of the NATM to the Underground Works at Tarbela," Water Power, Sep-Oct 1974.

Rabcewicz, L. v., Golser, J., and Hackl, E., "The Importance of Measurements in Cavity Construction," Der Bauingenieur, 47, 1972.

Rebull, P. M. and Gutierrez, R. G., "Shotcrete for Structural Rehabilitation," Use of Shotcrete for Underground Structural Support, Special Publication SP-45, 1974, American Concrete Institute, Detroit, Mich.